The Interrelationship of Material Toxicity, Stream Properties and Quantity of Spilled Material in Assessing the Risk of Hazardous Material Spills

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TAMU-SG-75-212
May 1976

TEXAS A&M UNIVERSITY SEA GRANT COLLEGE

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TAMU-SG-75-212

Partially supported through Institutional Grant 04-3-158-18
to Texas A&M University
by the National Oceanic and Atmospheric
Administration's Office of Sea Grants
Department of Commerce

$5.00

Order from:

Department of Marine Resources Information
Center for Marine Resources
Texas A&M University
College Station, Texas 77843

ABSTRACT

The current status of regulatory efforts for the bulk carriage of oil and hazardous materials is reviewed. The hazard posed to water resources is examined using the method of risk analysis. A weakness of the present regulatory efforts, namely that aquatic system properties and spill quantity are not considered, is identified.

With risk defined as the product of spill probability and severity, a procedure is suggested to better quantify one element of water pollution risk--severity of spill impact. The procedure identifies two major parameters which affect spill impact, concentration of material in the water and the concentration at which the material causes acute toxic effects. Methods are developed to quantify spill concentration in the water, a function of spill size and available dilution water, on a relative scale for use with existing relative toxicity ratings. The combined toxicity and dilution capacity ratings provide a significantly improved measure of water pollution impact, and thus risk.

The waterway relative dilution capacity quantification procedure is based on median discharge and considers the effects of longitudinal dispersion and the dynamics of fish mortality. The procedure is applied to the majority of the major inland and intracoastal waterways. Results are expressed as relative dilution capacity ratings for major waterway areas.

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CHAPTER I

INTRODUCTION

The terms oil and hazardous materials are used to describe a broad range of substances. Because of properties such as extreme reactivity, flammability, toxicity, and potential for severe environmental impact, substances in the OHM category usually require special precautions and handling. Common examples of such commodities are ethylene oxide, liquid chlorine, phenol, acrylonitrile, liquified natural gas, and ammonium nitrate. These products and many others are used in a wide variety of industrial operations, including the manufacture of plastics, fertilizers, and petrochemicals.

Oil and hazardous materials have played an increasingly important industrial role since 1945. One measure of their proliferation is evident in the ten-year growth pattern in industrial chemical shipments relative to 1958 shown in Figure 1.1. This growth pattern, coupled with increasing population concentrations in industrial urban areas as well as concern for the preservation of increasingly scarce and thus valuable environmental resources, results in a greater probability that major catastrophes will affect the public.

Objectives

The water pollution impact of an OHM spill depends primarily

The format of this dissertation follows the style of the Water Resources Bulletin of the American Water Resources Association.

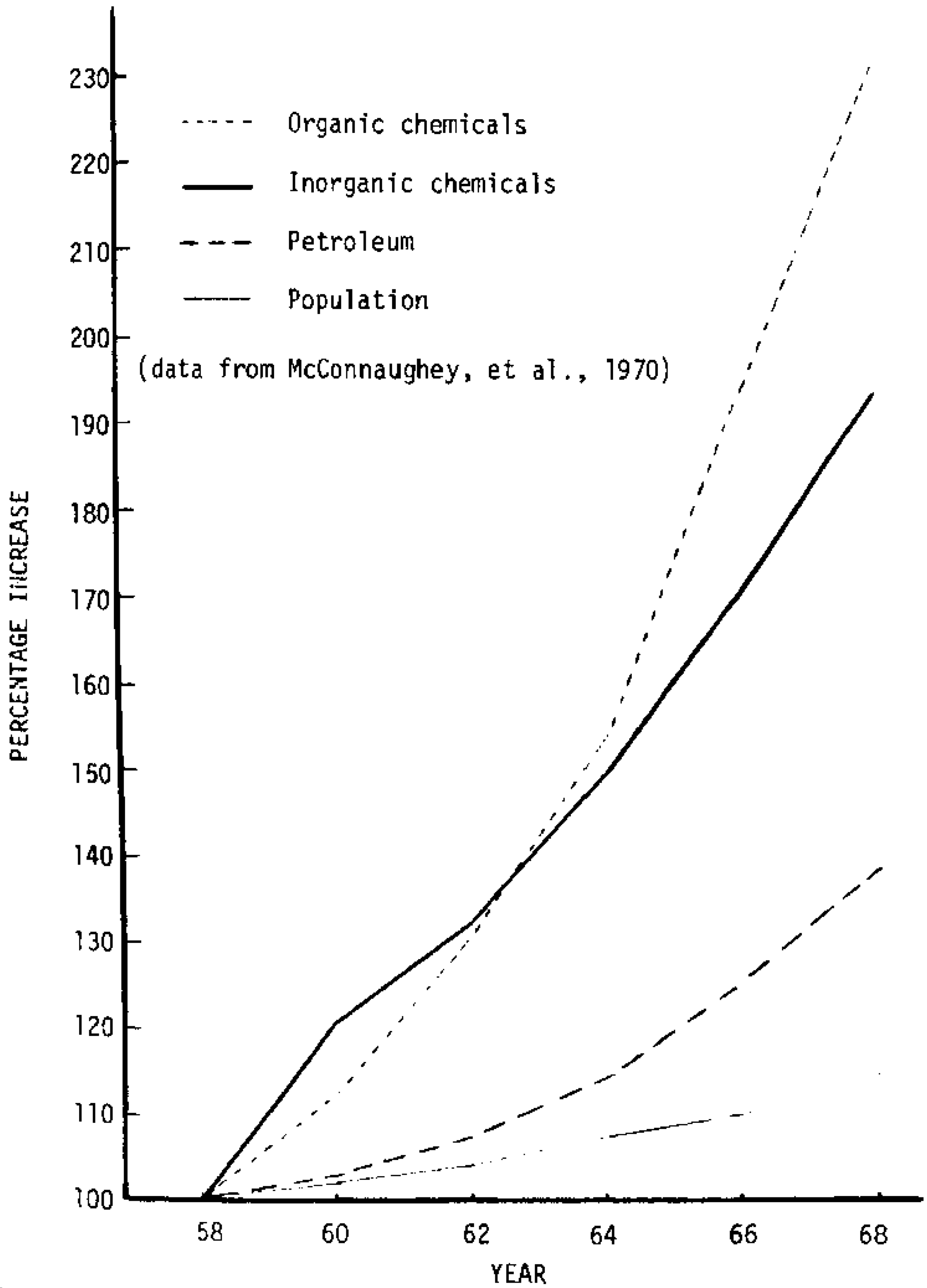


Figure 1.1. Percentage increase in total chemical shipments.

on the toxicity of the substance and the concentration of the material in the water. To date, the only consideration in transportation regulations regarding the water pollution impact of a spill has been the acute toxicity of the material. This research develops methods by which the probable concentration of a spilled material (a function of spill size and dilution capacity of the waterway) may also be considered in managing the water pollution risk. The benefit to be gained, i.e. more effective water pollution risk management, means that stringent and more expensive safety precautions may be applied selectively rather than across the board. By this selective application of additional safety precautions, the appropriate degree of safety required by the situation could be achieved, and an economic and environmental savings to industry and the public would result.

Procedure

Types of hazards posed by oil and hazardous materials will be investigated first, along with the historical development of the problem. Activities of government agencies dealing with the problem, as well as new techniques for managing hazardous material risks, then will be presented.

In Chapter II, a qualitative description is given of the wide range of OHM properties and their behavior in a spill situation. Chapter III investigates quantitatively the mixing of soluble materials in waterways, including a review of available dispersion modeling literature. The water quality aspects of OHM spills are investigated in Chapter IV, concentrating primarily on the problem of acute toxicity.

Analytical procedures to quantify, on a relative scale, dilution capacity, dispersion, and spill size are developed in Chapter V. In Chapter VI, these procedures are applied to the major U.S. Inland and Intracoastal Waterways. Results are presented in tabular form.

Types of Hazards

Oil and hazardous materials possess the capacity to harm people or the environment in several different ways. The following classification of hazard mechanisms, used by the U.S. Coast Guard, represents a reasonably complete overview of the types of threats posed by OHM.

1) Fire: the toll in lives, health, and property damage taken by fire is well documented. In addition to ease of combustion (usually measured by closed-cup flash temperature) and amount of heat per unit mass released upon combustion, other factors must be considered in rating materials as potentially hazardous. These factors include emmission of toxic fumes and dense smoke.

2) Reactivity: Three general types of reactivity hazards exist -- reaction with water, reaction with other chemicals, and self-reaction. The intensity of a chemical-water reaction depends strongly on such variables as temperature, chemical-water ratio and the amount of mixing. Reaction with other chemicals that may be stored in adja-

cent tanks depends on a similar set of circumstances, but is a much more complex problem because of the number of possible combinations of chemicals. Chemicals that undergo a hazardous self-reaction, may do so by rapid polymerization or oxidation.

3) Health Hazard: Liquids and gases present somewhat separate health hazards to personnel. A major hazard with liquids is direct contact, although there is some inhalation hazard since the substance may be released as an aerosol. Health hazards associated with direct contact are skin and eye burns. Air-transmitted toxicant effects must properly be considered as air pollution problems. Many substances can enter the atmosphere at a rate which constitutes a severe, acute air pollution problem. The quantities in which shipments are currently carried makes possible the exposure of very large areas to this acute toxic threat as may be seen in McConnaughey et al. (1970), where the hazard area from a single chlorine barge rupture is shown to cover the entire city of New Orleans. The air pollutant may be either a vapor or gas and may have widely varying dispersal characteristics, depending on the gas specific gravity, ambient temperature and atmospheric stability conditions.

In addition to short-term, acute hazards, many hazardous materials present long-term threats to personnel. For example, many chemicals are carried under cryogenic conditions and thus must be vented to avoid pressure build-up. This venting may be a significant cause of local air pollution problems. The problems resulting from venting may be either acute health threats, long-term threats such as from substances that are carcinogenic or mutagenic (vinyl chloride, for example), or economic problems that may be caused by corrosion of physical structures, damage to plant life, and property devaluations resulting from aesthetic problems such as odors.

4) Water Pollution: Many hazardous materials are extremely harmful if released into natural aquatic or marine systems. Water for domestic or industrial use may be rendered unfit, fish and other aquatic life may be damaged or killed, and recreational uses of waterways may be hampered by oily coatings, odor and color changes resulting from oil and hazardous material spills. The subject of water pollution effects of OHM spills will be discussed in greater depth in Chapter IV.

5) Radioactivity Releases: The potential damage of a release of these substances is generally appreciated. Effects range from acute radiation poisoning to a broad spectrum of sub-lethal effects including mutations and the possible inducement of cancer. The effects on an aquatic system can be especially severe if the radioactive material were to be incorporated in sediments, rendering an area uninhabitable for what may be a very long period. The use and carriage of radioactive substances is regulated by the Atomic Energy Commission (AEC).

Historical Perspective

The technology necessary to obtain and transport large quantities of potentially harmful material is a relatively recent development. When petroleum products first moved in commerce, they were handled using the best conventional technology then available. This resulted in some accidents. Shipping, however, was a risky business in those days, and it was realized that the development of new technology entailed additional risks. Gradually, safer operating procedures resulted. Risks were greater on tankers and munitions ships to be sure, but as Starr (1969) has pointed out, people are willing to accept substantially greater risks associated with employment than they are willing to accept as an innocent bystander. There were few problems obtaining crews for risky operations as long as the money and/or food were right.

A new dimension to the situation developed when it became apparent that innocent bystanders were subject to significant risks. This realization was somewhat slow in developing, however. In 1917, the port city of Halifax, Nova Scotia was severely damaged by the explosion of a munitions ship. Numerous smaller incidents occurred between the world wars and during the second world war, yet the time for significant action was not at hand.

The Texas City disaster of April 16, 1947, and the chlorine barge, WYCHEM 112, sunk in the Mississippi River near Nadres, Louisiana on March 23, 1961, had significant impacts in that they focused attention on the problem. The result of this attention was better shipping regulations and safer operating practices for ammonium nitrate and chlorine (NTSB, 1972). The broad problem of overall safety for innocent personnel and environmental resources was not approached. The problem may be briefly stated as: How is adequate safety assured for all parties involved, including the environment, at a minimum cost?

Traditionally, industry developed its own operating procedures for a particular substance. The standard of sufficiency in safety precautions was based on economic considerations rather than public welfare (NTSB, 1971). When the federal government first came into the area of OHM shipping control, the regulations used by the government were frequently the voluntary control measures in use by specific industries (NTSB, 1971).

The complexities of the OHM transportation regulation problem were examined by the National Transportation Safety Board (1971). Four principal difficulties with the existing regulatory programs were identified.

1) There was an absence of a clear, uniform objective in the Department of Transportation's (DOT) shipping regulations.

2) There was no technique for evaluating the full ramifications of proposed changes in regulations.

3) Discrepancies existed in the apparent levels of risk for different transportation modes and commodities.

4) There was no method of balancing the needs and interests of all parties affected, including innocent bystanders.

Acceptable risk levels were established on a case by case basis, frequently through the adversary proceedings of the Interstate Commerce Commission (ICC) (NTSB, 1971). In this procedure, parties-in-interest, the companies carrying the hazardous substance, and the ICC regulatory staff exchanged views in hearings prior to the establishment of regulations.

Although the interests of parties-at-risk, such as the public and the environment, were considered implicitly by the ICC staff, no formal procedure had been established by which these parties could have their interests advocated. The situation has resulted in substantial differences in risk levels from point to point in the transportation network.

Risk Analysis

The obvious need and the complex nature of the problem have prompted the development of several techniques to analyze and reduce risks. One of the earlier techniques for analyzing the failure of systems is Fault Tree Analysis, first developed by Bell Laboratories in 1962 for use in the aerospace industry. The undesired event is at the base of the tree. All pathways (limbs) leading to the undesired event are then drawn in, identifying each critical event along the path. The process of constructing the fault tree yields vital insight into the system being analyzed. With a knowledge of failure rates of individual critical components, it is possible to build in the redundancy necessary to make space flight, or hazardous material transportation, acceptably reliable and safe.

Another technique for analyzing risk was developed by Holmes and Narver (H&N) for the Department of Defense. It was used initially to analyze U.S. Army transportation of munitions. The H&N approach determines the overall risk, defined as the probability of an undesired event, such as an explosion, multiplied by the severity of the event should it occur. Probabilitites of accidents along sections of each transportation route are determined based on accident information. The severity of an accident along each section is determined by computing the range of effects such as blast damage or poison gas cloud radius and combining this with population density in each segment of the route to yield expected mortalities or injuries. The risk for each segment of the transportation route is then determined and summed over the entire route. This sum is then compared with other routes or modes and a minimum risk route selected.

A more sophisticated analysis technique was developed by Operations Research Inc. (ORI, 1973) for the U.S. Coast Guard. In this approach, each component of an accident situation (element on the Fault Tree) is analyzed in detail and the probability of a specific event computed based on such quantifiable parameters as vessel speed, maneuverability, hull plate thickness, channel conditions, etc. The effect of modifying a specific parameter such as stopping distance or double hull construction, then can be evaluated in terms of the overall reduction in the probability of the accident occurring. Further refinements of this approach are being pursued which include modelling the effects of a given accident so that a benefit-cost analysis can be applied to the whole range of transportation activities. With this tool, a realistic, quantifiable analysis of regulatory alternatives is possible.

Hazardous Material Regulatory Efforts

The hazardous material problem has elements which cross into the areas of several governmental agencies at all levels. The following brief review is intended to present merely a qualitative picture of the activities of these governmental groups, in order to provide the framework into which this investigation will fit.

The two main Federal Agencies involved are the Environmental Protection Agency (EPA) and the Department of Transportation (DOT). Under an inter-agency agreement, DOT has the responsibility for the prevention of accidents while in transit, and EPA has the responsibility for prevention of in-plant spills (Attaway, 1972). In addition once a spill threatens an inland waterway, EPA has the lead responsibility.

Environmental Protection Agency

The EPA's primary function is, as the name implies, the protection of the nation's environmental resources, including water. Under the 1972 Water Pollution Control Act Ammendments, EPA is authorized to designate hazardous polluting substances, to determine which are not subject to removal, and to assess fines or penalties for the discharge of these substances. These penalties can be up to $14 million for vessels.

To date, EPA's main thrust in the control of OHM releases has been 1) assisting industry develop better operating and safety practices so that the area outside the plant and the environment receive adequate protection, 2) developing counter-measure and clean-up technology (for examples see Marine Systems, 1971; Miller, et. al., 1973) and 3) restoring an area damaged by a spill.

Coast Guard

In addition to functions in vessel inspection, safety and rescue work, The U.S. Coast Guard has been very active in developing a means to cope with the problems of hazardous materials. One of their earlier efforts was to enlist the assistance of the National Academy of Science-National Research Council (NAS-NRC) in assembling experts on the various substances and hazards to recommend a system by which the Coast Guard could consider all these factors when they write shipping regulations. The first result of the NAS-NRC Committee on Hazardous Materials, NAS publication No. 1465, Evaluation of the Hazard of Bulk Transportation of Industrial Chemicals -- A Tentative Guide was published in 1966. This hazard evaluation system, as well as the latest draft revision (1974), consists of a hazard profile for each substance. The latest version has a numerical hazard rating (0-4) for nine specific hazard areas; fire, contact with skin and eyes, inhalation of vapors, inhalation of gases (short term), repeated inhalation of vapors and gases, water pollution hazard to humans through ingestion, hazard to aquatic organisms, reaction with water, and self-reaction.

Each hazard area has a numerical rating assigned, with 0 being the least harmful and 4 extremely harmful, based on a more or less specific test value for the substance. For example, the fire hazard is rated according to ranges of the closed cup flash point. The hazard profiles provide no information about how to regulate the commodity, but merely quantify on a relative scale, the inherent hazards associated with the substance.

Another area in which the Coast Guard has been working is the development of a fast response information system to aid personnel in responding to accidents. To provide the staff and facilities to allow the CG to respond effectively to spill situations, the Chemical Hazard Response Information System (CHRIS) was initiated. CHRIS consists of a staff based in CG Headquarters, armed with extensive information on the technical properties of the various substances and detailed plans of action. Coast Guard field personnel have available a condensed guide to chemical hazards which contains information such as notification procedures and immediate safety precautions. After on-the-scene personnel take the required immediate action, they contact the CHRIS main office with specific information on the accident such as chemical, size of spill, and the environmental circumstances. The main office, using their sophisticated information. including computerized chemical dispersion models and the area contingency plans, recommends specific spill response, control and clean-up techniques. Development of CHRIS was performed by A.D. Little Inc. (ADL, 1972) under a Coast Guard contract.

The National Contingency Plan (NCP) as revised in 1971 functions to coordinate federal agencies, local governments and industry to respond efficiently to emergency spill problems. The agencies primarily contributing to the plan are the EPA and the Departments of Transportation, Defense and Interior. Agencies advisory to the NCP are: The Departments of Commerce, Treasury, and Health, Education and Welfare and the Office of Emergency Preparedness (Hess, 1972).

The NCP sets up a National Response Team (NRT) whose function is planning and response organization, particularly where the emergency exceeds regional capabilities. Each region has an individual regional response plan and team. The regional plan provides information procedures for notification of key parties in the event of an accident, available resources to be used for containment, countermeasures, clean-up and disposal, restoration of affected areas, and procedures for recovery of damages and enforcement.

Other OHM Efforts

The Manufacturing Chemists Association, recognizing the risks associated with their products, has set up a Chemical Transportation Emergency Center (CHEMTREC) whose function it is to provide assistance in any transportation emergency involving chemicals. A toll-free number (800-424-9300), is manned constantly with experts ready to provide specialized advice for a particular substance.

The United Nations Joint Group of Experts on the Scientific Aspects of Marine Pollution (GESAMP) has the task of providing scientific advice to its sponsoring agencies (IMCO, FAO, UNESCO, WMO, WHO, IAEA) and the Intergovernmental Oceanographic Commission (IOC).

An Ad Hoc Panel of IMCO and GESAMP compiled a marine pollution profile for "noxious and hazardous" substances normally carried in bulk. The profile assigns ratings for hazard to human health, harm to marine organisms, or reduction of amenities or other uses of the sea. There are many similarities between this GESAMP hazard rating system, intended to advise the International Maritime Consultive Organization (IMCO) in such practices as deballasting and tank cleaning, and the NAS hazard rating system, devised to aid the U.S. Coast Guard in shipping regulations.

The Sea Grant Program of the National Oceanic and Atmospheric Administration, (NOAA), Department of Commerce, has, as its primary goal, the furtherance of wise use of coastal and marine resources. Noting the potential impact of OHM production and handling, as well as transportation, on the coastal zone, Texas A&M University as a Sea Grant college has sponsored several studies into the OHM impact on the coastal zone. These studies have included such activities as the environmental impact of an off-shore terminal for bulk liquid cargoes to the economic impact of OHM production facilities along the Texas Gulf Coast, as well as the present study.

Recent Proposals

One method proposed to limit the effects of hazards associated with OHM operations is to limit the size of cargo carried on one vessel. The question of whether this was a valid method to reduce the risk of OHM transport was approached for the Coast Guard by the NAS-NRC Committee on Hazardous Materials, Panel on Cargo Size Limitations. In its report (NAS-NRC, 1970), the Committee considered not only the increased impact of larger shipments but also the greater degree of safety afforded by having fewer larger vessels operating, and the reduced probability of accident that would result from better containment and safety practices possible by taking the economies of scale made possible from larger vessels, such as better navigation equipment and fewer and better trained personnel. The Panel concluded that it did not appear valid at that time to restrict shipment of OHM cargoes on the basis of size alone.

The overall problem complexity was approached by Danahy and Gathy (1973) in a risk reduction format. They proposed that risk be quantified on a relative rather than an absolute basis and that this relative measure of safety then be compared to a measure of safety required for a particular area. They note that safety is a complex function of a number of variables and propose representations of these functions which take into account the major variables while ignoring those they consider to be of less significance. They consider the problem as consisting of three areas: the cargo, the vessel, and the exposed area such as the port/terminal/waterway.

The cargo is assigned a cargo index (CI) which is determined from semi-empirical formula for each hazard mechanism. For example, the suggested CI for flammable clouds was

$$CI = 10\ K_1\ (\rho_V/TLV)^{0.5}\ , \qquad (1.1)$$

where ρ_V is the specific vapor density (ppm), TLV is the threshold limit value (ppm), and K_1 is a relative vaporizing value (dimensionless) which is a function of Reid vapor pressure. Similar type formula exist under Danahy and Gathy's method for each of the modes of hazard such as flammable clouds, explosion, radioactivity, oxidizing potential and water pollution hazard. The specific hazard mechanism which yields the largest value for CI is the one which is employed.

Next, a vessel safety index (VI) is computed with a similar type formula which takes into account those factors which influence vessel safety. For example such factors as increased total tank capacity, higher speed, a larger turning circle, and outside location in a tow would tend to decrease VI, while such things as double-hull construction, number of subdivisions in the hull and an inside location in a tow would tend to increase vessel safety and VI. These factors are combined by Danahy and Gathy into a formula which is a relative measure of vessel safety.

The CI and VI are then combined into a transportation safety index (TSI) which is defined as VI/CI. Greater relative safety is reflected in a higher value of TSI. The TSI is then compared with a measure of the safety requirements of the port/terminal/waterway or port safety index (PSI). The PSI takes into account such factors that would contribute to an accident and factors which represent the degree of impact should the accident and release of cargo occur. The first group of a factors includes such things as channel width, visibility, channel turn radius and current velocity, while the second group includes such factors as population density both fixed and mobile, and industrial facilities such as tank farms and warehouses in the vicinity of the transportation route. The PSI is dependent strongly on local conditions such as weather, river stage, and whether any major population density such as a sports stadium in the risk exposed area. Danahy and Gathy suggest methods by which the PSI can be reduced. These include such things as limiting transit to times of daylight and good visibility, controlling traffic or providing a tug escort to the hazardous material shipment.

If the TSI is greater than the PSI, transit would be permitted. If, on the other hand, TSI is less than PSI, some method must be employed to change the situation. Any of several methods could be used to do this such as carrying the hazardous material barge under separate tow, thus increasing maneuverability parameters, and thus VI, or reducing the PSI through special escort or choice of transit time.

While the approach proposed by Danahy and Gathy (1973) offers several advantages such as greater consistency of risk exposure and allows greater flexibility to industry and the Coast Guard to meet the requirements of transportation safety, there are the obvious disadvantages of complexity and the difficulties that might arise over the development of suitable hazard index evaluation schemes. It would also require a large enforcement bureaucracy to evaluate the various risk indices in each step of the transportation network. It remains to be seen whether this will be the risk control effort of the future.

As Danahy and Gathy (1973) point out, many variables determine overall system safety. While it may not be practical at this time to include all of these variables in a risk reduction program, it is possible to include major parameters. As needs and the required technology become more developed, it may be possible to include more factors.

The present efforts at minimizing environmental risk from the carriage of OHM center mainly on shipping regulations for the substances. These regulations control such things as container specifications, marking and shipping practices. These regulations are based on the properties of the substance itself (one source of this information is the hazard profiles of the NAS Committee on HM) and not on the the vulnerability of the area in which the transportation takes place. An exception to this statement would be for transportation of certain extremely hazardous substances in some congested areas.

Damage to water resources, however, is a function not only of the acute toxicity of a material, but also of the concentration of the substance in the water, as well as a number of other factors such as the type of organisms exposed, water hardness, and temperature. While acute toxicity is perhaps the most significant of these variables, the concentration of pollutant resulting from a spill, which is a function of spill size and waterway dilution capacity, is a close rival.

This can be seen by noting that of the 362 substances supplied by the Coast Guard to the NAS Committee on Hazardous Materials, 93% had aquatic toxicity hazard ratings ranging over three orders of magnitude. The total range of the NAS aquatic toxicity scale is from LC_{50} greater than 1000 ppm to less than 1 ppm, although some substances exceed this scale by a good margin. The range of dilution water discharge available on American Inland Waterways varies from the hundreds of cubic feet per second (cfs) to hundreds of thousands of cfs or approximately three orders of magnitude. In addition, sections of intracoastal waterway have essentially no flow except for tidal oscillations.

Although the techniques are not now available to quantify accurately all parameters which affect aquatic system damages, it is possible to determine with reasonable accuracy the dilution capacity of American Inland and Intracoastal Waterways. This research proposes to analyze flow statistics of these waterways and to present a technique whereby dilution capacity can be quantified and employed as a parameter to reduce the risks faced by waterways from shipments of oil and hazardous materials. Such a system would be a first step toward the more complete solution to the problem as proposed by Danahy and Gathy (1973), yet it would be more readily implemented. Advantages of such a system in terms of increased control of risks to coastal and inland waterway systems would be substantial, in that the more vulnerable regions could receive the additional protection required without penalizing shippers in the aquatic systems which are more capable of withstanding a spill of OHM.

CHAPTER II

OIL AND HAZARDOUS MATERIAL SPILLS--A QUALITATIVE DESCRIPTION

The umbrella term, oil and hazardous materials, includes substances with a broad range of physical and chemical properties. Because of the range of physical and chemical properties, the process of dispersion in the environment and eventual removal may differ greatly between substances. This chapter will examine the physical and chemical characteristics of OHM and the relation of these characteristics to the spill hazards posed to the environment. The techniques of analytical modeling of these general classes of materials will also be reviewed.

Every substance spilled into a waterway will eventually be reduced to harmless levels through a combination of mechanisms such as dilution, sorption onto sediments, chemical or biological decay, or evaporation. Sorption and chemical or biological decay may be important removal methods for some materials, but in this analysis of dilution capacity they will not be considered.

[illegible]

Many materials which are gases at ambient conditions are carried in liquid form either under pressure or reduced temperature or both. Examples of this type of substance are liquid anhydrous ammonia (LNH_3), liquified natural gas (LNG), and chlorine. If these substances are released above the water, they will disperse primarily as a gas. In this case, the hazard may be acute inhalation toxicity (as with chlorine) or fire (LNG). Models for predicting the dispersion of an atmospheric pollutant as a function of wind speed and stability conditions have been developed but are not the subject of this study.

Frequently, liquified gases are soluble in water, and if released under the water surface, will enter solution in varying amounts. In a study of the release of LNH_3 for the Coast Guard by A.D. Little (1974), it was found that for surface spills the amount of LNH_3 entering solution ranged between 65 and 82%, and was relatively independent of spill quantity or rate, water salinity, air temperature, and air and water motion. When the release was below the water surface, the percentage entering solution increased to 91-95%, depending strongly on depth of release. Once the soluble gas enters solution, its behavior is that of a solute and will be discussed in depth in the next chapter. In general, the water pollution hazard posed by gases is reduced by the evaporation process.

Highly Volatile Liquids

Substances which do not boil at ambient temperatures, but still evaporate rapidly, are the next major category. The reduction in water pollution hazard due to high evaporation rate depends on ambient water temperature and the solubility and specific gravity of the material. If the substance or the resulting solution is more dense than water, high evaporation rate will be much less of a factor. If, on the other hand, the substance remains close to the surface, evaporation will remove the material more quickly than would the normal dilution process. Again, the water pollution hazard is reduced when a substance evaporates rapidly and is less dense than water. Examples of such materials are cyclohexane, methyl ethyl ketone, and gasoline.

Insoluble Substances Less Dense Than Water

These materials spread over the water surface. Water pollution effects resulting from this class of materials include damage to wildlife through external coating or direct toxic action, inhibition of natural reaeration of the waterway, and restriction of recreational and water supply uses. Perhaps more important than the water pollution concern is the severe threat posed to personnel by fire and direct toxic fume emission.

Initial spreading of a substance over the water surface is determined primarily by gravity, viscosity and surface tension (Ichiye, 1972).

Techniques for modeling insoluble substances other than oils are in the early developmental stages (A.D. Little, 1972). Wind and current data necessary to evaluate long-term diffusion of the spilled material are difficult to assimilate into a predictive model. For these reasons, insoluble substances less dense than water are not considered in the waterway dilution capacity analysis.

Insoluble Substances More Dense Than Water

These materials present a somewhat unusual set of hazards to the environment. Because they sink to the bottom of a waterway and are relatively insoluble, they are quickly removed from sight. They tend, however, to be concentrated along the bottom where they can have severe effects on benthic life. Examples of such materials are ethylene dichloride, trichloroethane, and dibutyl phthalate.

The impact of spills of low solubility, dense materials has been given secondary importance (Dawson et al., 1970). In many cases, the solubility of the material is lower than its reported toxic concentrations, indicating a much reduced threat to aquatic life. This statement, however, cannot apply without question to all substances whose solubility is listed as "nil" in a chemical handbook. For chemical purposes, less than 1 percent solubility may be in fact nil, yet this concentration (10,000 ppm) is quite high compared with reported LC_{50} values for the majority of industrial chemicals on the NAS hazardous materials list.

Motion of insoluble, dense materials is through the combined action of initial momentum, gravity flow (density current) and the stress produced by water motion over the material. Except in the case of steep bottom slope and low water velocity, movement of the spilled substance tends to be somewhat slower than the surrounding water. There is also a tendency for the material to remain pooled in low areas such as impoundments before dams.

Considerable work has been done with the behavior of buoyant plumes, often in conjunction with studies of ocean brine outfalls for desalination plants. Flume studies at the U.S. Army Corps of Engineers Waterways Experiment Station for the Office of Saline Water (1971) document the behavior of highly saline water injected into fresh water. Empirical relations are presented which describe the lateral spread and dilution of the saline plume. Abraham (1965) investigated horizontal jets in stagnant fluids and determined relations which describe the position, velocity and concentration of the axis of the jet in terms of initial velocity.

More closely related to the behavior of a spilled, dense substance is the density current. Fietz and Wood (1967), in a laboratory study of a saline density current, observed that there was very slight vertical spread of the saline plume, even at the lowest density difference ($\frac{\Delta\rho}{\rho} = 0.01$), a turbulent flow regime, and a high bottom slope angle (20.0 degrees). Horizontal spread angles were much larger, indicating a tendency of a dense substance to spread horizontally to the banks of the waterway, yet stay very close to the bottom. In spite of these results, very little research has been directed toward predicting the motion of insoluble substances along actual streambottoms.

While all of the general classes of OHM are important to water pollution problems resulting from spills, the data and modeling techniques are not available for use in an analysis of spill dilution on waterways. For this reason, waterway dilution analysis will be limited to the behavior of solutes.

CHAPTER III

MIXING OF SOLUTES IN OPEN CHANNEL FLOW

In motionless fluid, the spreading of a solute is described by the Fickian diffusion relation

$$\frac{\partial C}{\partial t} = K \nabla^2 C \quad , \tag{3.1}$$

where C is concentration, t is time, and K is a molecular diffusion coefficient dependent on fluid properties.

When a uniform fluid velocity u, is superimposed on the spreading solute, the diffusion relation becomes

$$\frac{\partial C}{\partial t} = -u \nabla C + K \nabla^2 C \, . \tag{3.2}$$

Under turbulent conditions, with no mean vertical or transverse velocity components, equation (3.2) becomes

$$\frac{\partial C}{\partial t} = -u \frac{\partial C}{\partial x} + \frac{\partial}{\partial x}(e_x \frac{\partial C}{\partial x}) + \frac{\partial}{\partial y}(e_y \frac{\partial C}{\partial y}) + \frac{\partial}{\partial z}(e_z \frac{\partial C}{\partial z}) \, , \tag{3.3}$$

where e_x, and e_y and e_z are, in the longitudinal, transverse and vertical directions respectively, the turbulent or eddy diffusivity coefficients obtained by time averaging the equations of motion and solute concentration. In general, these coefficients will not be constant as is the case with molecular coefficients, as they are functions of fluid flow properties. The molecular contribution to diffusion may be effectively ignored under most conditions.

As Taylor (1953, 1954) noted, the primary method of mixing a solute in pipe flow was through boundary-induced velocity differences in the flow dispersion. Before proceeding further, it is worthwhile to discuss and define, for the purpose of this work, the various mixing and transport processes.

Diffusion, Dispersion and Convection

The terms diffusion and dispersion are sometimes confused and often used imprecisely. Although general agreement exists on their meanings, certain overlapping areas may be found. Fundamental to both dispersion and diffusion is convection -- the transport of a solute across a boundary in the flow at the same velocity as the fluid. Diffusion is transport of a solute across a boundary by molecular scale or larger scale turbulent motion. Dispersion is transport produced by variations in velocity across the boundary.

Holley (1969) has proposed that diffusion refer to transport in a given direction at a point because of the difference between true convection in that direction and the time average of convection in that direction. Dispersion refers to the difference between true convection and the spatial average of the convection in that direction. In the case of longitudinal dispersion, the spatial average would be taken across the stream cross-section.

A somewhat clearer and certainly easier to remember distinction was given by Fischer (1968 a) when he likened diffusion of a solute to the random motion of a drunkard's walk and dispersion of the solute to the non-random effect of the inebriated traveler's being released along a busline (stream line in a river) and traveling different distances at different rates.

In this work, transport by essentially random processes will be referred to as diffusion and transport by differences in velocity across the stream cross-section will be termed dispersion. Dispersion is the dominant diluting mechanism in natural streams (Fischer, 1967). Where the various transport mechanisms are combined into one coefficient, it is termed a dispersion coefficient.

With the release of a soluble, neutrally buoyant pollutant, mixing and dilution proceeds, both by diffusion and dispersion, in all three directions. As noted by Aris (1956), and discussed by Fischer (1966, 1967), concentration profiles are skewed from a Gaussian distribution in this early stage of mixing. As channel boundaries are encountered by the spreading pollutant, cross-sectional concentration distributions approach uniformity. At this point, dispersion of a pollutant slug was noted by Taylor (1953, 1954) to be adequately represented by a one-dimensional Fickian diffusion equation.

The rate of pollutant spreading to the channel boundaries will be discussed first, followed by a discussion of the contributions of Taylor to the understanding of one-dimensional dispersion processes. Methods of predicting dispersion coefficients in the one-dimensional representation will then be reviewed.

Vertical Mixing

As will be discussed in greater depth, the dominant mixing mechanism in open channel flow is velocity shear. Using a logarithmic velocity distribution, Elder (1959) determined eddy viscosity, assumed to be equal to the eddy diffusivity, to be determined by

$$e_z = 0.068\ h\ u^*, \tag{3.4}$$

where h is depth of flow and u* is shear velocity, which for open channel flow is given by $(hgs)^{1/2}$, where g is the acceleration of gravity and s is the slope of the energy grade line.

The vertical transport of pollutants in open channels was investigated by Jobson and Sayre (1970). The behavior of both neutrally buoyant fluid and discreet negatively buoyant particle pollutants in two-dimensional, uniform flow was described using a finite difference solution. The effects of different velocity profiles and mass transfer coefficients on predicted concentration profiles were analyzed and found to be relatively small. They found a depth averaged vertical turbulent diffusivity, e_z, to be given by the Elder (1959) relation with the constant equal to 0.067.

Crickmore (1972) studied dispersion in a shallow open channel leading to the port of Heysham, England. Neutrally buoyant radioactive tracer, discharged just below the surface at low nozzle velocity, was used. A vertical array of radiation detectors was employed in transversing the plume. There was essentially no density stratification in the channel. Vertical mixing was essentially complete (maximum depth ~13.0 m) within 500 m of the discharge point with tidal current velocities ranging from 0.35 to 0.70 m/s (.75 - 1.35 kts).

Vertical, compared to transverse and longitudinal, mixing is a relatively rapid process. Stewart (1967) in the lower Mississippi found that vertical mixing from a surface release of dye was complete well before the dye had moved to its first sampling point 22 miles downstream, while transverse mixing was still not quite complete.

Transverse Mixing

The same mechanism is responsible for transverse as well as vertical mixing, but since the shear-induced turbulence is not isotropic, no direct relation exists between the two. A number of laboratory flume studies of e_y cited in Fischer (1973), indicate e_y/hu^* to range between 0.15 and 0.20.

In a straight irrigation canal, Fischer (1967) measured a higher value, $e_y/hu^* = 0.23$, but attributed this to the thalweg of the flow meandering from side to side in the canal. Elder (1959), however, measured the same value in his much smaller scale laboratory flume studies.

In large-scale field studies on the Missouri, Yotsukura, et al., (1970), report values of e_y/hu^* of 0.6. Similarly, Glover (1964) reports values of 0.72 in the Columbia River. These higher values more than likely reflect increased transverse mixing induced by meanders in the stream (Fischer, 1973).

Ward (1973) analyzed the time required to achieve complete cross-sectional mixing. Using equation (3.3) and the method of images for impermeable boundaries, Ward determined when pollutant concentration was uniform to within $\pm$ 5%. Ward's results indicate that the time or distance to achieve 95% mixing is strongly dependent on the point where the tracer was injected. If the tracer is injected only 10% of the transverse distance off the center of flow, the distance to achieve the same degree of mixing is increased by a factor of 2.4.

With a similar aim, to determine when cross-sectional mixing was complete so that a one-dimensional representation would be valid, Fischer (1967) defined a time scale for dispersion $T' = l^2/e_y$, where l is the characteristic length of the cross-section, taken as the distance from the point of maximum velocity to the farthest bank. Through flume studies he determined that the Taylor model was valid at a distance given by

$$L > 1.8 \frac{l^2 \; u}{r \; u^*} , \qquad (3.5)$$

where r is hydraulic radius.

In Fischer (1973), this result was expressed in terms of dimensionless time defined as

$$T'' = T \frac{.23 \; h \; u^*}{l^2} . \qquad (3.6)$$

For the Taylor period to apply, T'' must be greater than 0.4.

For example, a channel 300 feet wide (l=150'), 15 feet deep, $u^* = 0.1$ feet per second, $u = 1.0$ feet per second, the time to achieve complete mixing would be less than eight hours.

The Contributions of G.I. Taylor

Taylor, (1953, 1954), working with pipe flow in both laminar and turbulent ranges, observed that after a sufficiently long time the cross-sectional distribution of a slug would be nearly uniform. When this happened, the shearing action, tending to produce vertical or transverse concentrations gradients, would be exactly balanced by the vertical or cross-sectional diffusion. This balanced process produced an apparent Fickian diffusion along the channel axis with the coordinate system moving with the mean flow.

Taylor's (and Fick's) dispersion equation is

$$\frac{\partial C}{\partial t} = -u \frac{\partial C}{\partial x} + D \frac{\partial^2 C}{\partial x^2} \quad , \tag{3.7}$$

where D is a dispersion coefficient which includes the effect of longitudinal diffusion. In Taylor's work, D was taken as essentially constant. The dispersion coefficient can be evaluated from tracer data by noting the rate of change of variance, σ_x^2 , with time (Diachishin, 1963; Fischer, 1966) often termed the change of moment method.

The analytical solution to equation (3.7) under conditions of constant cross-sectional area and dispersion coefficient and no source or sink terms is

$$C(x,t) = \frac{M}{A} \frac{\exp}{(4\pi Dt)^{\frac{1}{2}}} -\left(\frac{\{x - ut\}^2}{4Dt}\right) , \qquad (3.8)$$

where M is mass of material and A is cross-sectional area.

Taylor's analysis involved development of a limited form of equation (3.7) in cylindrical coordinates, and then integration of this form to determine the dispersion coefficient. Laboratory experiments then confirmed the theoretical representation. Elder (1959) followed a similar analysis except in Cartesian coordinates. Taylor and Elder's method of solving for D in Cartesian coordinates is as follows:

A turbulent velocity and concentration field is defined in the usual manner, that is, the velocity at a point is the sum of a mean value signified by an overbar and a turbulent fluctuation from the mean signified by a prime (') e.g. $u = \overline{u} + u'$, $C = \overline{C} + C'$ etc. Turbulent transfer coefficients as previously referenced, are defined by the Reynolds analogy as $\overline{u' \ C'} = e_x \frac{\partial C}{\partial x}$ etc.

A coordinate system $\xi = x - u\,t$, moving with the mean velocity is also defined. With the following assumptions, these values are substituted into equation (3.7).

$$\text{a)} \quad \frac{\partial C'}{\partial t} << \frac{\partial \overline{C}}{\partial t} \qquad \text{b)} \quad \frac{\partial C'}{\partial \xi} << \frac{\partial \overline{C}}{\partial \xi}$$

$$(3.9)$$

$$\text{c)} \quad \frac{\partial C'}{\partial \xi} = \text{constant} \qquad \text{d)} \quad \frac{\partial \overline{C}}{\partial t} = 0$$

Equation (3.7) can then be expressed as

$$u' \frac{\partial \bar{C}}{\partial \xi} = \frac{\partial}{\partial z} \quad e_z \frac{\partial C'}{\partial z} \quad . \tag{3.10}$$

This can be integrated over the vertical coordinate, z, to yield an expression for C'.

$$C' = \frac{\partial \bar{C}}{\partial \xi} \int_0^z \frac{dz}{e_z} \quad \int_0^z u' \, dz \tag{3.11}$$

If the mass transfer of solute across a section of area A at can be described by a coefficient times the mean gradient of the solute

$$\int_A u'C' \quad dA = - DA \frac{\partial \bar{C}}{\partial \xi} , \tag{3.12}$$

then the dispersion coefficient can be solved for

$$D = -\int_0^z u' \, dz \int_0^z \frac{dz}{e_z} \int_0^z u' \, dz. \tag{3.13}$$

Using a logarithmic velocity profile

$$u(\frac{z}{h}) = - \frac{u}{k} \quad \log (1 - \frac{z}{h}), \tag{3.14}$$

where k is von Karman's constant equal to approximately 0.4, Taylor and Elder performed the integrations in (3.13) and found D to be given by

$$D = \gamma \, r \, u^* . \qquad (3.15)$$

In Taylor's pipe flow analysis, γ was found to be 10.1. In two-dimensional (longitudinal and vertical) flow, Elder found a similar relation with h substituted for r and the coefficient γ was 5.86. Laboratory investigations by both Taylor and Elder confirmed (3.15).

Longitudinal dispersion Coefficient Prediction Relations

Thackston and Krenkel (1967) examined the problem of determining dispersion coefficients from flow parameters in both laboratory and uniform field conditions. They found that in uniform two-dimensional (vertical) flow conditions, the predictive relation of Elder (1959) was reasonably valid. They determined a minor improvement to (3.15) through regression analysis which was

$$D = \gamma \, h \, u^* \left(\frac{u}{u^*}\right)^{1/4} , \qquad (3.16)$$

with γ in the range of 5.0 to 8.0, as was Elder's. Thackston and Krenkel also concluded that non-uniform flow or the presence of bends or other discontinuities render the use of (3.16) invalid.

Similar dispersion coefficient prediction relations were obtained analytically by Sooky (1969) for two particular types of channel geometries, triangular and circular. The form of these relations is the same as (3.15) with γ determined by cross-sectional geometry. Sooky notes that his relations do not apply in non-uniform flow conditions.

Attempts to apply (3.7) to natural streams with D predicted by (3.15) have met with little success. Godfrey and Frederick (1970, reprint of 1963 study), found that although the one-dimensional model was applicable, dispersion coefficients were underestimated by a factor of 4 to 35. Similarly, Glover (1964) found D to be predicted in natural streams by (3.15) but with γ equal to 500. Later, Yotsukura, Fischer and Sayre (1970) in the Missouri River at Sioux City found γ to be in excess of 5600. Obviously, (3.15) does not completely describe the longitudinal mixing process in natural streams.

The most probable explanation for these differences, aside from measurement before equation (3.7) was valid, was that non-uniform flow, bends and dead zones in the flow in natural streams frequently combined to give a much greater mixing rate than would be achieved in the idealized conditions of the laboratory flume. As a result of these differences, much research has been directed to understanding the natural stream dispersion process. As a part of this effort, a great deal of field work has been directed to achieving usable, directly measured dispersion coefficient values. These measurements have been made for a variety of reasons such as determining the waste assimilative capacity of a waterway, the effects of thermal inputs or time of travel studies.

Where dispersion coefficient measurements have been made in a waterway at a certain discharge, and D is desired at a different discharge, a technique was presented by Fischer (1967) to allow for change in discharge. Fischer (1967) expresses qualitatively the dispersion coefficient derived from the Lagrangian time scale as

$$D \propto \overline{u'^2} \; l^2 / r \, u^* \, . \qquad (3.17)$$

For natural channels, Fischer notes that (3.17) can be used to provide an estimate of the change in dispersion coefficients with discharge.

The observed discrepancy between predicted Gaussian concentration distributions and the sharp rise and slow die-off observed in natural streams (Godfrey and Frederick, 1970; Fischer, 1967) was investigated by Thackston and Krenkel (1967) and Thackston and Schnelle (1970). Using a dead zone model first proposed by Hays

$$\begin{aligned} \frac{\partial Ca}{\partial t} &= Da \frac{\partial^2 Ca}{\partial x^2} - u \frac{\partial Ca}{\partial x} + Ka\,(Ca - Cd) \\ \frac{\partial Cd}{\partial t} &= Kd\,(Ca - Cd) \, , \end{aligned} \qquad (3.18)$$

in which the subscripts a and d refer to the main stream and dead zone areas respectively, and the K's are volume-based mass transfer coefficients, Thackston and Schnelle (1970) indicate a better fit between model prediction and reality. Difficulties with using (3.18) are that the solution involves either a La Place Transform form given by Hays or a modified version given by Thackston and Schnelle (1970) and additional data on concentrations in the main stream and dead zone.

A solution to (3.18) developed by Thackston, Hays and Krenkel (1967) is

$$C_i/(M/Au) = \left(\frac{Pe}{4\pi t_i t_{ave}}\right) \exp - \left(Pe \frac{(1-t_i/t_{ave})^2}{4\ t_i/t_{ave}}\right), \quad (3.19)$$

where Pe is the Peclet number ux/D, t_{ave} is mean residence time x/u, and M/Au is the area under the concentration curve. Using observed values C_i and t_i, a non-linear, least-squares curve-fitting technique is used to determine D. The authors claim the non-linear technique is a significant improvement over other existing methods of longitudinal dispersion coefficient determinations.

In a theoretical investigation of the effects of boundary irregularities on the effective dispersion coefficient, Okubo (1973) determined an asymptotic value for the effective longitudinal turbulent diffusivity to be given by

$$B = \left(\frac{1}{1 + r'} + \frac{r'}{(1+r')^3} \frac{u^2}{Ak'}\right) D'' \quad , \qquad (3.20)$$

where B is the effective longitudinal turbulent diffusivity, D" is the longitudinal turbulent diffusivity determined without irregularities, r' is the ratio of trap volume to stream volume and k' is the reciprocal of the average residence time in the trap. Using oscillating flow in the Mersey River as an example, with $r' = 0.01$, $k' = 10^4$, $D'' = 0.93 \times 10^5$ cm^2/sec, and U = 150 cm/sec, this relation gives B to be approximately ten times greater than D''.

Fischer (1968b) published a paper detailing what he calls a routing procedure for determing D from dye dispersion data. Fischer notes long tails of dye frequently follow the slug passage. These tails make a large difference in measured variance thus making dispersion coefficient determination difficult with the change in moment method alone. Fischer's routing method determines D initially by the change in moment method, cutting off the tail at an arbitrary point. Using this value of D, a predicted concentration profile is produced which is then compared with the measured profile at the same point. The dispersion coefficient is then adjusted to minimize the mean square error between measured and predicted concentration profiles. The result of Fischer's routing technique is a "best" value for D, providing of course, that the one-dimensional model is applicable.

Using an analysis technique similar to Taylor (1953, 1954) and Elder (1959), Fischer (1967) presents a numerical method of longitudinal dispersion coefficient prediction in natural streams. Fischer's method involves neglecting velocity variations in the vertical since most natural streams have greater horizontal than vertical velocity variations. Using Fischer's relation

$$D = -\frac{1}{A} \int_A u' \, dA \int_0^y \frac{dy}{e_y h(y)} \int_0^y \int_0^{h(y)} u' \, dz \, dy \,, \quad (3.21)$$

where e_y is the lateral turbulent diffusion coefficient found by Elder (1959) to be given by $e_y = 0.23\ h\ u^*$ and h(y) is the depth as a function of horizontal position, it is possible to evaluate directly the longitudinal dispersion coefficient at a point from a knowledge of cross-sectional geometry, shear velocity u^*, and the cross-sectional distribution of velocity variation, u'. This relation was used by Fischer (1968a) on the Green and Duwamish Rivers, Washington, and found to be reasonably effective.

Bansal (1971) compiled the results of a number of workers in developing one and three-dimensional representations of the dispersion equations that considered the effects of vertical and lateral dead zones, dye adsorption and decay. These equations also included a regional dispersion factor whose nature was determined experimentally. Bansal analyzed a number of natural streams of varying characteristics and determined empirical predictive relations for longitudinal as well as three-dimensional dispersion coefficients. An example of his equation is for a dimensionless dispersion coefficient

$$\log \left(K'' \frac{V_s\ D}{V\ u^*\ h}\right) = 6.467 - 0.714 \log \left(\frac{u^*\ h}{\nu}\right), \quad (3.22)$$

where K" is Bansal's regional dispersion coefficient, V_s is effective mean velocity of flow at sampling station, Q/A, and V is the observed velocity of the maximum concentration point in the stream.

Boning (to be published, 1974) analyzed U.S. Geological Survey time of travel studies, many of which are unpublished. These dye studies were examined using multiple correlation techniques to fit the data to empirical relations which predict various parameters for a slug load of pollutant in terms of readily available quantities such as discharge, channel slope and reach length. These relations were determined for channel controlled streams, and riffle and pool reaches. Accuracy presented for these relations ranges from 26 to 50 percent.

McQuivey and Keefer (1974) approach the problem of determining longitudinal dispersion coefficients from a somewhat different tack. They note a similarity between the one-dimensional dispersion equation (3. 7) and a linearized one-dimensional flow equation which they give as

$$\frac{\partial q}{\partial t} = - c \frac{\partial q}{\partial x} + K^* \frac{\partial^2 q}{\partial x^2} , \qquad (3.23)$$

where q is discharge per unit width, c is the advective velocity, and K* is a flow dispersion coefficient. Using empirical relations between c and u and between D and K* the authors present a method for estimating the longitudinal dispersion coefficient from mean flow parameters

$$D = 0.058 \; Q_0 / S_0 W_0 , \qquad (3.24)$$

where Q_0, S_o and W_0 are initial discharge, slope and width respectively. This relation, used where the Froude number $F = u / gh < 0.5$, was checked against Fischer's (1968 b) routing procedure for 18 streams and found to give reasonably accurate results, the standard error of estimate being 30.0 percent.

Drawing on the work of Orlob (1959), Callaway, et al. (1969) used a dispersion coefficient predicted by

$$D_i = \text{Const. } E^{1/3} \; l_i^{4/3} , \qquad (3.25)$$

where E_i is energy dissipation given by $E_i = u_i \; g^{\Delta H/} L_i$, l_i is the scale of the mixing (a function of hydraulic radius), $\Delta H/L_i$ is difference in potential head at the ends of the channel, g is gravitational acceleration and u_i is the mean velocity in segment i.

The effect of bends on longitudinal dispersion was investigated by Fischer (1969) and by Fukuoka and Sayre (1973). Fischer's approach was to extend his method (1967) of dispersion coefficient prediction to the curved flow case using a radial velocity distribution developed by Rozovskii. The algorithim developed was demonstrated to predict accurately dispersion coefficients on the Green-

Duwamish and Missouri Rivers. Fischer notes that bends affect longitudinal dispersion in two ways: by concentrating the area of high velocity to the outside of a curve, longitudinal dispersion is increased; by inducing secondary eddies in the bends, the rate of transverse mixing is increased which tends to reduce longitudinal dispersion. The effect of alternating direction of the bends was found to be important where the ratio of cross-sectional mixing time to the time to flow around an individual curve was large.

In the analysis of the effects of bends given by Fukuoka and Sayre (1973), an attempt is made to relate more easily obtained channel properties to observed longitudinal dispersion coefficients. One of several semi-empirical relations presented which appears to function well over a fairly broad range of natural stream data presented is

$$D = u^* h \left(\frac{W r_c^3}{L^2 h^2}\right)^{0.86}, \qquad (3.26)$$

where r_c is the average bend radius, W is width, and L is the bend length. In Figure 3.1 the supportive data presented by Fukuoka and Sayre (their figure 7a) is presented along with dispersion data obtained from time of travel studies of the U.S. Geological Survey. Although additional scatter is evident, the new data follow the trend of the Fukuoka and Sayre relation closely.

Equation (3.26) has the advantage of not considering channel

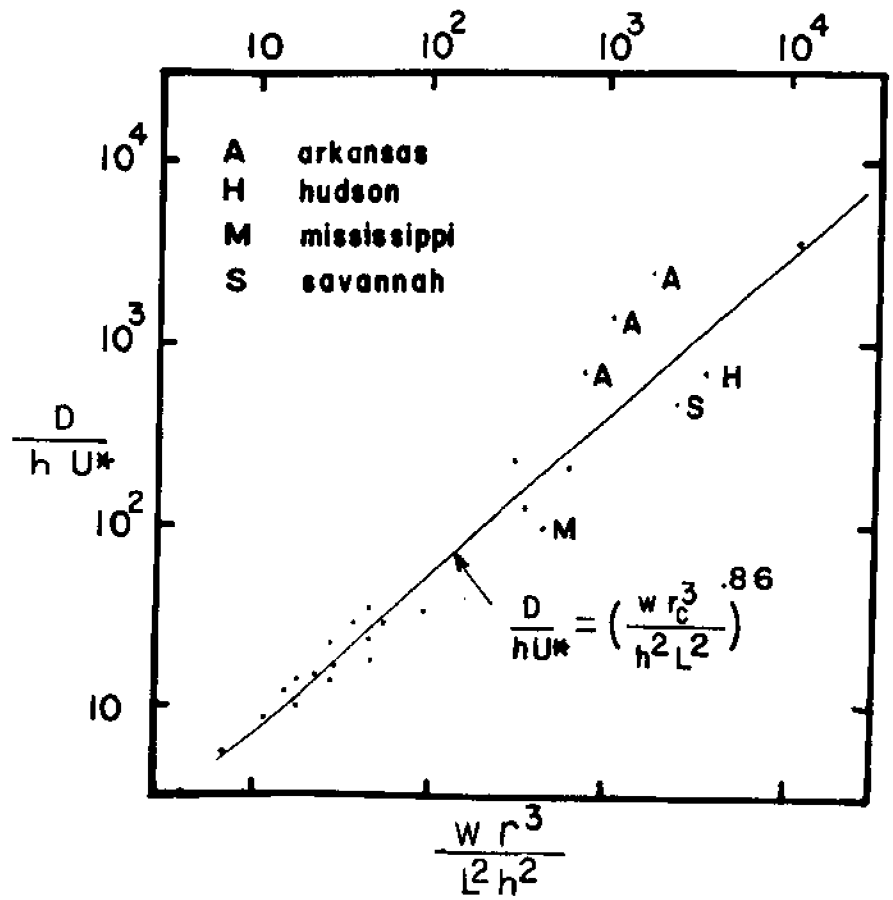


Figure 3.1. Relation of dimensionless dispersion coefficient to average channel properties.

slope explicitly. In many cases, particularly at low flow conditions in regulated waterways, channel slope information is quite difficult to obtain.

Fukuoka and Sayre caution that the empirical relations they present are based on a limited data sample and should be used with caution. One obvious limitation is that they do not apply (indicate infinite D) in straight channels.

Equation (3.26) indicates a larger D where channel bends are gentle. The physical explanation of this is, as noted by Fischer (1969), that the effect of bends is two-fold. One effect is to concentrate high velocity flow to the outside which increases shear and thus dispersion. The other effect is that small rapidly alternating bends tend to mix high velocities across the cross-section, reducing velocity shear and dispersion.

In the actual spill situations, the concentration distribution depends strongly on the duration of the spill. In a theoretical analysis, Yen (1970) has shown that the rate of decay of initial concentration, C_0, to a specified peak concentration $C_p(t)$ is related to the duration of injection of pollutant. Using Yen's information, the ratio $C_p(t)/C_0$ can be stated as

$$C_p(t)/C_o = \frac{X}{(4\pi Dt)^{\frac{1}{2}} + X} \qquad (3.27)$$

where X is the length of stream water contaminated by the pollutant, equal to $u\ T_d$ where T_d is the duration of the dump. For an example, with u = 2 ft/sec, D = 1000 ft^2/sec, the ratio $C_p(t)/C_0$ after two days for a one-day discharge is .79, while for a one-hour discharge, the ratio is .13.

As noted previously, longitudinal dispersion is a function of velocity shear in the stream cross-section. The stream velocity field is determined by several factors including cross-sectional shape, meanders and mean stream velocity. With a detailed knowledge of the velocity field, direct prediction of longitudinal dispersion is possible (Fischer, 1967). Without a detailed knowledge of the velocity field, dispersion may only be inferred from bulk or average parameters.

Of the numerous relations presented in this review, only that of Fukuoka and Sayre (1973), equation (3.26), considers all of the above mentioned factors in a fashion which is readily useable with available data. In addition, this predictive relation has been verified with reasonable accuracy for most of the dye dispersion data now available on larger waterways. For these reasons, equation (3.26) is used to estimate D where measurements are not available.

Estuarine Longitudinal Dispersion

While the equations describing the spreading of a spilled solute also apply in estuarine flow, there are significant differences which must be considered. These include periodic changes in velocity corresponding to changes in the tide and two-and three-dimensional circulation patterns resulting from density differences, wind stress and coriolis accelerations. Where system complexity, data availability and computer capacity are present, multidimensional numerical models have been developed (Reid and Bodine, 1968; Hann and Young, 1972). Nevertheless, the one-dimensional model similar to equation (3.7) is still widely used where conditions allow.

As noted by Harleman (in Ward, 1971), estuarine models may be classified according to their time scale. If ΔT is of the order of minutes, a large portion of the spectrum of fluid turbulence is considered in the model while if ΔT is the tidal period, only the effects of the advective fresh water flows will be considered. The latter is the so-called slack-tide approximation (Stommel, 1953; O'Connor, 1965) where the effects of tidal velocity induced mixing are incorporated into a different and larger dispersion coefficient.

Holley, et al. (1970) has shown that dispersion in oscillatory flow is dependent on the ratio, T', of the tidal period to the time for cross-sectional mixing. They note that the dispersion coefficient varies as the square of this ratio for $0 < T' < 1$ and that $T' > 1$ the dispersion coefficient is essentially constant and equal to a value corresponding to the average hydraulic conditions during the tidal cycle.

For wide estuaries, in the transverse direction, generally $T' << 1$, and the effect of transverse velocity distribution on the dispersion coefficient decreases with increasing width. In the vertical, however, T' is normally greater than unity thus making vertical velocity variations the dominant mechanisms for dispersion as in the Taylor/Elder formulation.

Holley and Harleman (1965) determined that the longitudinal dispersion coefficient could be assumed constant in oscillating flow if the time average of the magnitude of the velocity was used in the Tqylor's relation. After converting u* to a flow parameter and a resistance coefficient and applying a factor of 2 to account for the effects of channel bends, their dispersion coefficient relation becomes

$$D = 100\ n\ U_{max}\ r^{5/6}, \qquad (3.28)$$

where n is Manning's friction coefficient and U_{max} is the maximum velocity during the tidal cycle.

The difficulties of estimating a dispersion coefficient that is constant over the entire tidal cycle have been avoided by solving continuously for the mean velocity of flow. This has been accomplished by simultaneous solution of the continuity and momentum equations to obtain water surface elevation and discharge as functions of time. This technique has been used by Harleman and Lee and by Daily and Harleman (both in Ward, 1971) to obtain good representations of flow in an estuary. The disadvantage of this approach is the increased computer time required.

A typical estuary may be thought of as possessing two regions: a fresh water tidal region and a salinity intrusion region. The discussion so far has only applied to the fresh water tidal region and to the salinity region where complete vertical and horizontal mixing of the salt water intrusion has occurred. Where strong vertical or horizontal density variations exist at the mouth of estuaries, the conditions required for a one-dimensional model are violated, and the best that can be expected is approximation obtained by an artificially large dispersion coefficient. For example, Stigter and Siemons (in Ward, 1971) found that the dispersion coefficients obtained by matching observed salinity distributions were as much as two orders of magnitude greater than would be obtained by the Taylor approximation.

Open Water Diffusion

Where solute concentration is unbounded by a land interface, the rate of dilution is solely a function of density structure and the intensity and shape of the turbulence spectrum. Oceanic turbulence is in general much stronger in the horizontal than in the vertical due to suppression effects of the air/sea interface and a boundary of large vertical stability (thermocline). Because of this, the horizontal aspect of diffusion is more important in most cases. An example of this is given in Folsom and Vine (1957) where a radioactive tracer spread over 40,000 km^2 in 40 days while remaining in the mixed layer less than 60 m deep.

The horizontal mixing process is primarily controlled by the amount and scale of horizontal turbulence. Turbulent eddies much larger than the solute plume produce advection of the entire plume. Scales of turbulence much smaller than the plume are the mechanism of turbulent diffusion. These smaller eddies cascade down in scale to molecular diffusion. Eddies of the same scale as the solute plume produce meanders and other irregular shapes associated with a dispersing plume. All the turbulent scales, except the very largest, contribute to the mixing of the plume. A knowledge of the turbulent structure is required before a reasonably accurate prediction of dilution rate can be made. This estimate is generally quite difficult to obtain in coastal and open ocean areas.

Some success has been achieved using both types of models although the results of different workers in different areas are far from uniform. A more complete discussion of oceanic diffusion may be obtained in Pritchard et al. (1971).

CHAPTER IV

WATER POLLUTION FROM HAZARDOUS MATERIAL SPILLS

Oil and hazardous material spills present a full spectrum of water pollution problems, including the poisoning of water supplies, damage to the natural aquatic system and rendering the waterway unsuitable for recreational uses through aesthetic problems such as color and odor. Poisoning of water supplies and aesthetic problems, while certainly serious, are relatively straightforward. These parameters either meet criteria or they don't. Aquatic systems, however, are a much more complex subject because the organisms affected vary widely in species and tolerance and damage to one specie may affect others far removed from the spill scene. Damage to aquatic systems may proceed through such mechanisms as: 1) direct lethal toxicity 2) sub-lethal disruption of activities of the organism 3) incorporation of the substances into the tissue of the organism and 4) changes in the habitats of the organisms.

Direct lethal toxicity refers to interference with cellular or sub-cellular processes leading directly to death. Sub-lethal effects are also disruptions of cellular processes but are those which do not lead directly to death. However, physiological and behavioral processes may be affected by these disruptions which could produce death at a later time. Activities where the disruptions may be especially critical are feeding and reproduction.

The uptake of substances in the tissue of organisms presents problems through: 1) tainting of the flesh, affecting commercial value, 2) "biomagnification", where accumulation of the substance up the food chain results in harmful concentrations and 3) human health hazards due to accumulation of carcinogenic substances, particularly polycyclic aromatic hydrocarbons (PAH), in the flesh of many organisms (Zobell, 1971).

Habitat changes can render survival of organisms impossible by altering the nature of their environment. For example, insoluble substances whose specific gravity is greater than the ambient water can change sediment properties. Rapid oxidation of a spilled substance can deplete dissolved oxygen to below an organism's survival level.

Finally, damage to the aquatic community through any of these mechanisms can quite possibly produce synergistic effects with other damage mechanisms, thus compounding the problem.

The complexity of these water pollution problems, and the vast number of variables they involve, make it difficult to include each factor in an overall hazard model. Moore (1973), working with crude petroleum, has proposed a quantified summary which considers the solubility and effects of the various hydrocarbon fractions of a typical crude on various aquatic flora and fauna. Hann (in NAS-NRC, 1974) demonstrated a qualitative relation between aquatic system hydraulic properties, spill quantities and probable effect. The complexity and lack of methodology to predict accurately such parameters as amount of material in solution and the type of organisms present in a given spill situation have, however, limited regulation efforts to consideration of relative acute toxic effects. There is a need for better OHM risk management (NTSB, 1971). This research seeks to extend the available management parameters to include receiving waterway hydraulic properties.

Acute Toxicity

Acute toxicity is commonly considered to be toxic effects or irreparable damage occurring within a short time, while chronic toxicity deals with long-term exposure to a toxicant at a level which does not produce immediate effects. The dividing line for laboratory testing between acute and chronic toxicity is not well defined. In studies on rats and other small mammals, two weeks is typically taken as the limit for acute effects (Smyth and Carpenter,1948). In studies on the effects on aquatic life, the interval typically chosen ranges from one to seven days, with four being the most common. The choice of time interval is very dependent on the substance being tested, test organism and environmental conditions of the test.

The word toxicity also involves a range of considerations. Toxic effects can be defined as occurring at the level of observable symptoms, first observed death, a given percentage mortality or complete mortality. In any given population of organisms, some will be very vulnerable to the toxicant, while others will be very resistant.

It has been observed that the most replicable indication of toxicity is the dosage at which 50% of the organisms have died or have been incapacitated in a specified time. For this reason, and because continuous observation of the organisms is not required, the most common form for reporting acute toxicity is now the dosage which results in 50% mortality within a specified time (LD_{50}).

The procedure for determining LD_{50} values is to give a range of toxicant dosages to groups of test organisms and observe percent mortality at the end of the specified period. Percent mortality versus dosage is plotted, and the dosage which produces 50% mortality is determined by any of several curve-fitting techniques (Harris, 1959; Bliss, 1937).

Where the test organisms are mammalian species such as rats, the dosage is given by any of several routes, e.g. orally or intragastrically (ig), intraperitoneally (ip), intravenously (iv), subcutaneously (sc) or by some other route. Dosages, scaled to test organism body weight, are commonly expressed as milligrams substance per kilogram body weight (mg/kg). In the case of oral administration, the test is termed oral LD_{50}. For aquatic test organisms dosage is controlled by concentration in the test dilution water. The test may be either a batch test, where the concentration is set initially and uncontrolled during the test, or a continuous flow test, where pollutant concentration is adjusted continuously to maintain a constant concentration. The tests are termed static or continuous flow LC_{50}, or TLm (Tolerance Limit, median).

While this type of toxicity test may be the most consistent, serious deficiencies still exist. The results of an LC_{50} test on one organism in one type of water will not necessarily be applicable to another organism, water type, stage in the organism's development, or temperature of the water. An example of this is the substance aniline ($C_6H_5NH_2$) where five reported toxic concentration using different test organisms and laboratory conditions produced values that ranged over two and a half orders of magnitude (Hann and Jensen, 1973).

Acute toxicity information in the form of LD or LC_{50} data, though useful, particularly in regard to the relative toxicity of the substances, provides little predictive information about the effects that could be expected from a spill situation. A controlled laboratory experiment considers a finite number of organisms, test concentrations, durations, and environmental conditions, while an actual spill situation contains a full range of these conditions. In addition, the full tabulation of effects involves a level of understanding of ecological interaction which is now being only approached in some instances. It is easy to see why caution is often counseled in the application of laboratory results to field conditions.

While caution is certainly appropriate, the problem of hazardous material spills demands that the best available information be used in the analysis of alternatives open to decision makers. Acute toxicity information in the form of LD_{50} and LC_{50} information forms the basis for evaluation of water pollution effects in the NAS and GESAMP water pollution hazard rating systems. Laboratory data are essentially the only source of information on the effects of spills in different aquatic systems.

Acute Toxicity Threat to Personnel

Certainly the most important aspect of the problem of hazardous material spills is the safety of human beings involved. As has been recognized by all groups considering the problems of hazardous materials, a spill of a potentially toxic or hazardous substance has the capacity to poison people far removed from the scene through their consumption of contaminated water.

Recognizing this danger, the U.S. Geological Survey (in NAS-NRC, 1970) developed a simple technique for estimating a maximum concentration of a conservative pollutant if the amount spilled and discharge of the stream are known. This relation,

$$C_{max} = \frac{8.0}{T_L} \times \frac{M}{Q} , \qquad (4.1)$$

where C_{max} is in ppm, M is weight of spill in pounds, Q is discharge in cfs and T_L is travel time, was developed from fluorescent dye data from USGS time of travel studies. Using this relation, and a knowledge of the acute toxicity of the spilled substance, authorities at the spill scene can estimate the degree of hazard posed to down-stream communities and recommend action such as the temporary shut-down of municipal water systems.

The criterion used is the maximum concentration in the stream compared to the acute oral LD_{50} value of the substance. This criterion applies reasonably well to the hazard faced by the human population. Exposure will likely be of such short duration as to closely approximate the situation of a single dose oral acute toxicity test.

The criterion is modified in some cases where the toxic action of a substance is different in dilute solutions than in the concentrated solutions used in laboratory tests. This and similar factors are, however, considered in the human toxicity hazard ratings of the NAS and GESAMP Systems.

Acute Toxicity Threat to Aquatic Systems

In the case of aquatic toxicity, natural organisms are exposed to a variable concentration over widely differing durations. The critical factor may well be duration of exposure rather than the maximum concentration to which an organism is exposed. Certainly the two factors both play an important role in determining the fate of the exposed aquatic community. It is important, therefore, to investigate this relationship.

Literature on toxic action on aquatic organisms will be briefly reviewed, and a mathematical expression developed to model toxic action in a varying concentration spill situation. Using the mortality model developed, the effect of major waterway hydraulic variables such as mean velocity and longitudinal dispersion, will then be investigated. The aim of the investigation is to determine the most effective manner, in terms of acute toxic effects, to characterize the concentration distribution of a spilled substance.

In any given population of organisms, some will be very vulnerable to the toxicant and others will be very resistant. While the relation between percent mortality and time varies widely with toxicant, test organism and test conditions, the general relation can be expressed by the family of curves in Figure 4.1. For examples see: Brown et al., 1969; Buhler et al., 1969; and Lammering and Burbank, 1960; Herbert and Merkens, 1952. In general, time between spill and first mortality decreases at higher concentrations. At very low concentrations, depending on the organism, few if any organisms are killed. Between these two extremes lies a concentration at which only a small percentage of test organisms die if the period of exposure is short, while a high percentage are killed if exposure is long enough. Exceptions do exist to these general statements, but they are relatively rare and do not restrict use of this analysis.

If percent mortality after a specified time is plotted versus concentration, the form of curve shown in Figure 4.2 results. Many

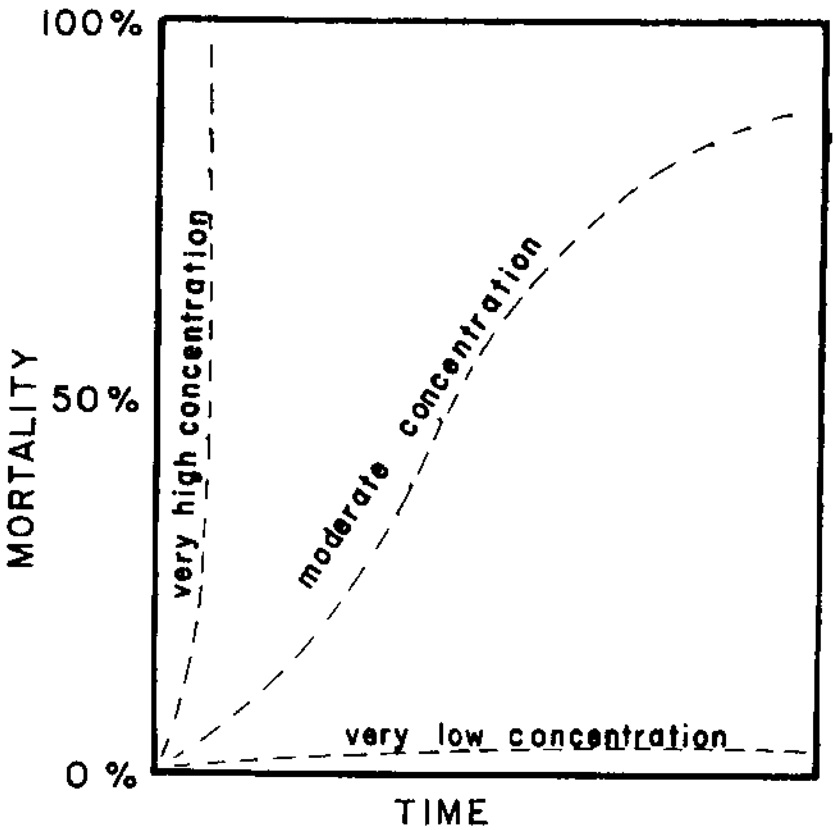


Figure 4.1. General relation of percent mortality to time at various toxicant concentrations.

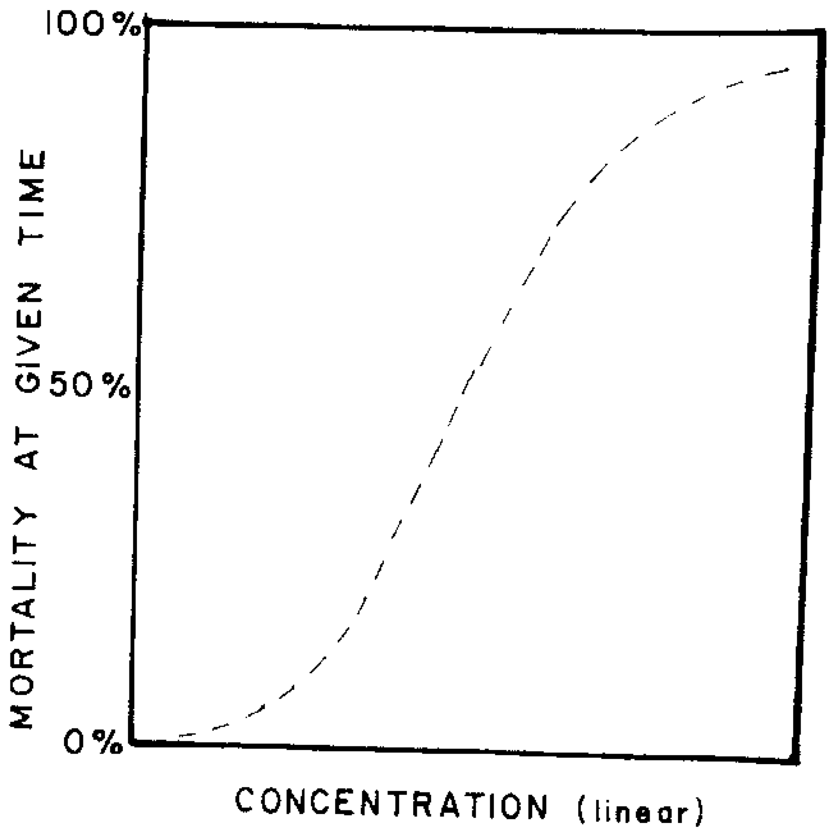


Figure 4.2. General relation between percent mortality and toxicant concentration at a given time.

workers (Herbert and Merkens, 1952; Lloyd, 1960; Chen and Selleck, 1969; Lammering and Burbank, 1960; Jones, 1964) have noted that if the log of median survival time is plotted versus log concentration, a curve is obtained which is often linear over much of its range. This suggests that the relation between concentration and time to a given percent mortality might be expressed by

$$C^n \; T = K' \;, \tag{4.2}$$

where n and K'are empirically fitted constants, C is concentration, and T is the time to the given percent mortality, usually 50%. Wuhrmann and Woker (1950) and Burdick as cited in Jones (1964), both noted that a concentration and a time exist below which no toxicant-induced mortality occurs and suggested a refinement to allow for these variables

$$(C-C_t)^n \; (T-T_t) \;=\; K' \;, \tag{4.3}$$

where C_t and T_t are threshold concentration and reaction time, respectively. Figure 4.3 from Wuhrmann and Woker (1950) illustrates this relationship. Although this relation must be true at extreme values of C and T, variations may result from the smooth curve predicted by equation (4.3). C_t, T_t, n and K'may take on a broad range of values depending on toxicant, test organism, and environmental conditions.

Working with anthrax spores and several disinfectants at fixed

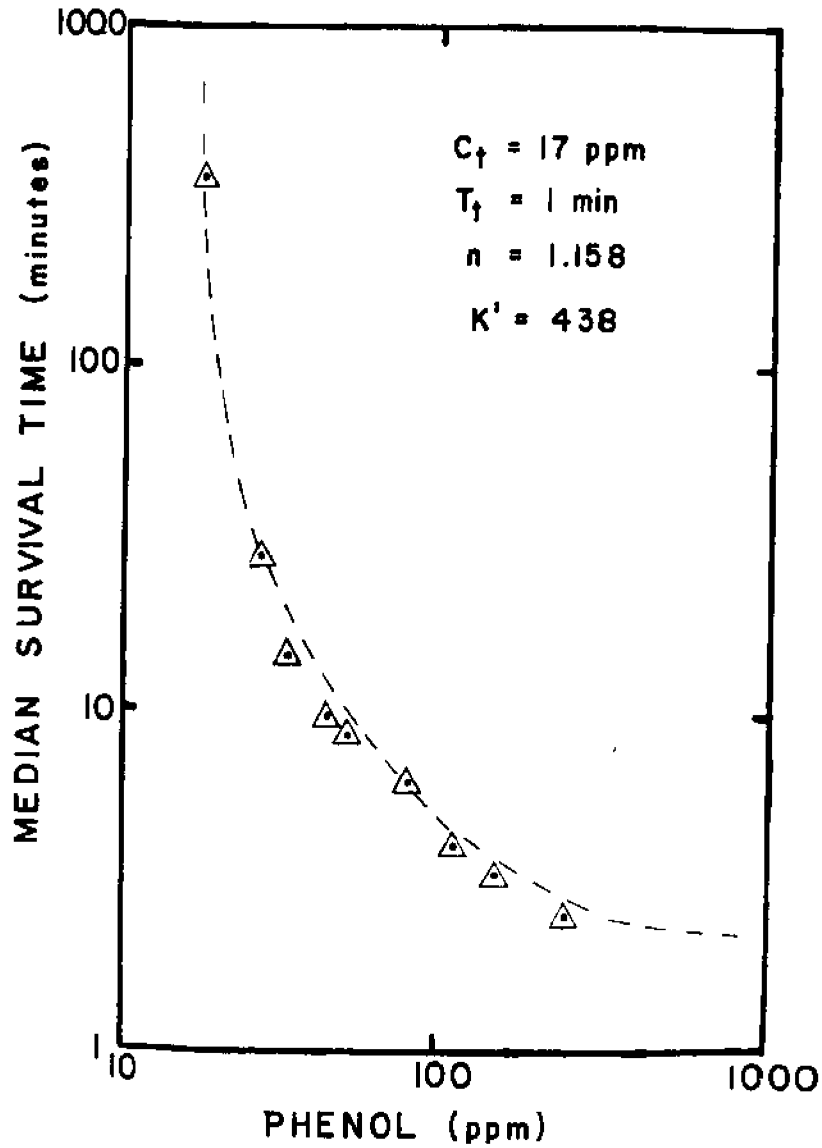


Figure 4.3. Concentration of phenol versus median survival time for minnows (Phoxinus laevis Ag). (Data from Wuhrmann and Woker, 1950.

concentrations, Chick (1908) observed that the rate of mortality in a given system is related to organism population. This relation

$$\frac{dN}{dt} = -KN \, , \tag{4.4}$$

where N is the number of organisms present and K is an experimentally determined rate coefficient, is now referred to as Chick's Law. Data on a variety of higher organisms and toxicants presented by Chen and Selleck, 1969; Brown, et al., 1969; and Lammering and Burbank, 1960, confirm the general applicability of equation (4.4).

The rate of kill, K, varies with toxicant, concentration, organism and a range of environmental considerations such as temperature, water hardness, dissolved oxygen level, etc. In a particular spill situation, however, K is exclusively a function of concentration as the other parameters do not vary greatly. The rate of kill as a function of concentration may be evaluated through measurements of K over a range of concentrations. The experimental efforts involved in these measurements are much more severe than those required for LC_{50} tests and, as a result, these data are not available for a large number of substances, test organisms and conditions. There are sufficient data, however, to evaluate the general shape of this function, and to compute its values for specific instances.

Equation (4.3) suggests that there is a maximum value for K in equation (4.4) which will yield a given percent mortality in the threshold time (T_t), however high the concentration. Below the threshold concentration, C_t, K must drop to zero since the concentration is low enough for the organism to remove the toxicant as rapidly as it is received.

These facts suggest an exponential form for K:

$$K = \alpha\,(1 - e^{-\beta(C-C_t)}), \qquad (4.5)$$

where α is the maximum value of K, and β is an experimentally determined constant that governs the rate of change of K with concentration. Figures 4.4, 4.5, 4.6, 4.7 show the relation of this function to data presented by the indicated authors and values of α, β, C_t, and T_t. Departures from this form of relation have been observed where very low concentrations of pollutant have toxic effects if duration of exposure is sufficiently long (Herbert and Merkens, 1952). These departures indicate that K should be a sigmoid rather than exponential curve. As the primary thrust of this research is on short-term exposure, these departures are not considered. With K determined by equation (4.5), it is theoretically possible to evaluate percent mortality in a systen where toxicant concentration is constantly varying.

Based on available experimental evidence, equation (4.5) adequately reflects the dynamics of population mortality and will be used in this work. It is realized, however, that this model has not been experimentally verified in a varying concentration situation.

Evaluation Procedure

As noted earlier, the threat to personnel consuming contaminated

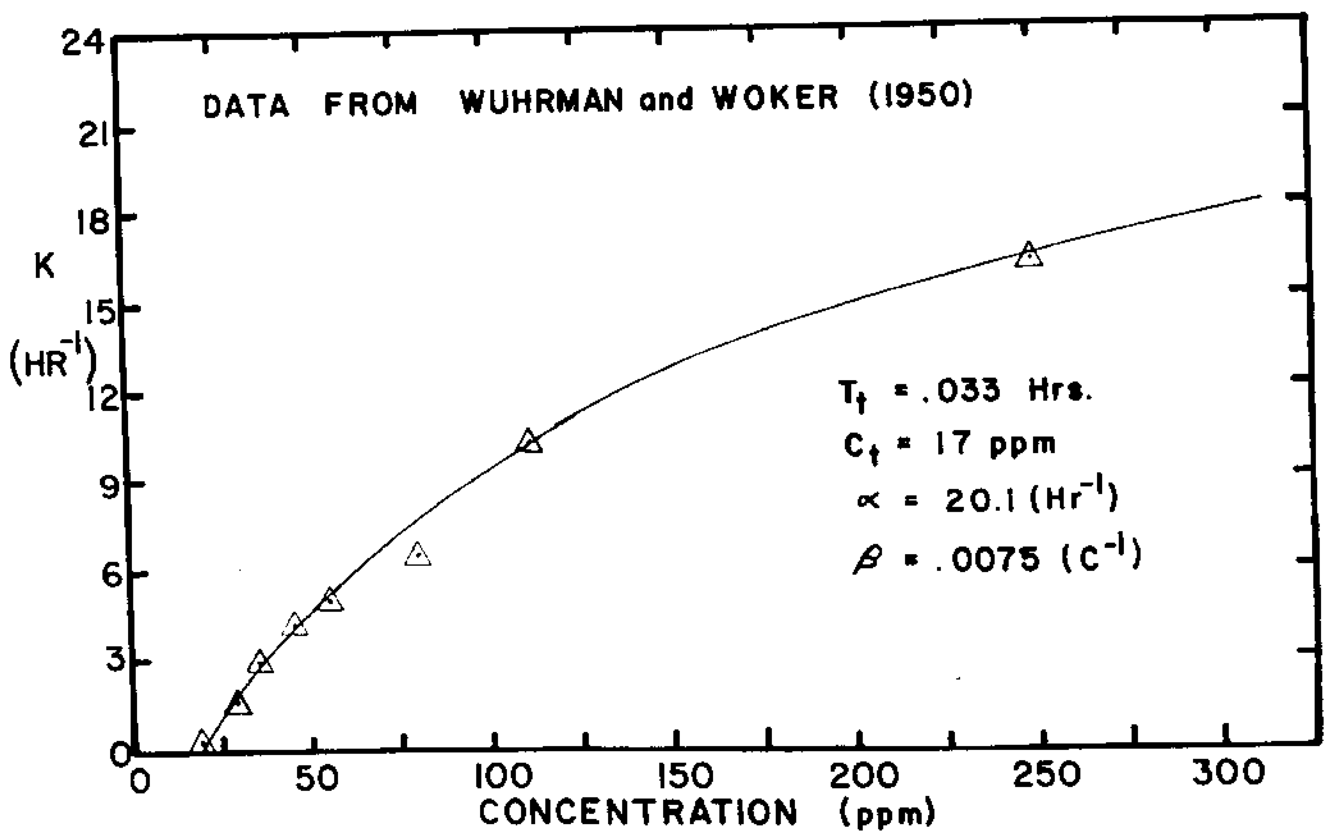


Figure 4.4. Mortality rate coefficient for phenol on minnows.

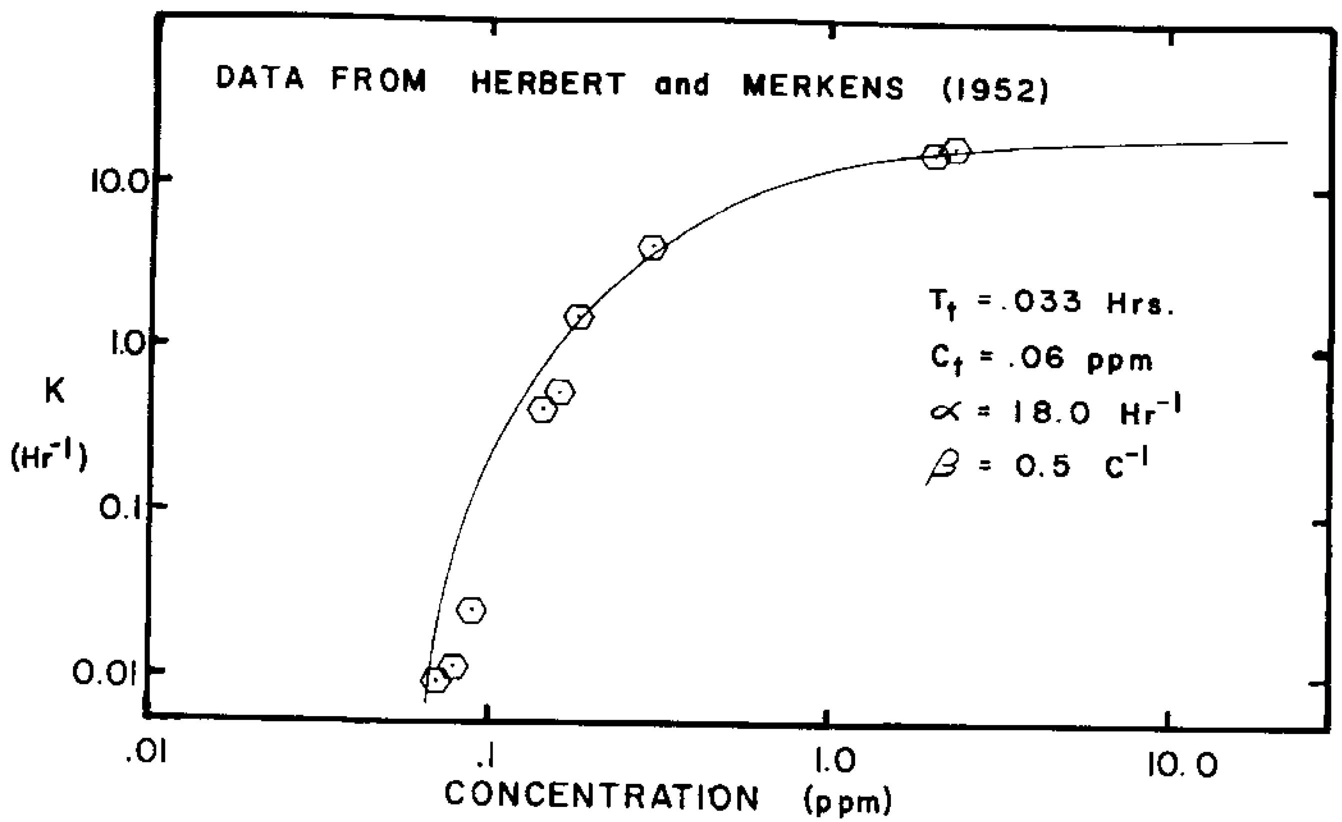


Figure 4.5. Mortality rate coefficient for cyanide on trout.

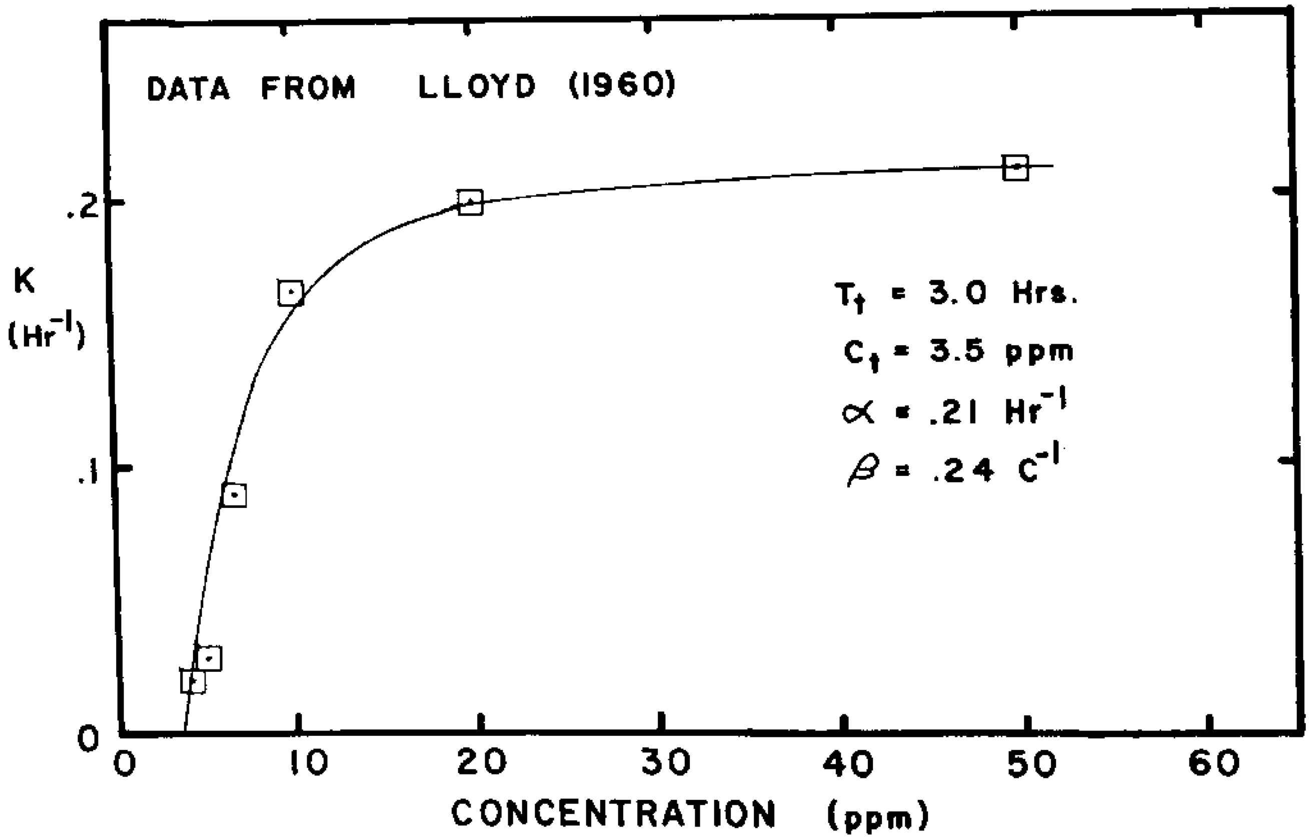


Figure 4.6. Mortality rate coefficient for zinc sulfate on trout.

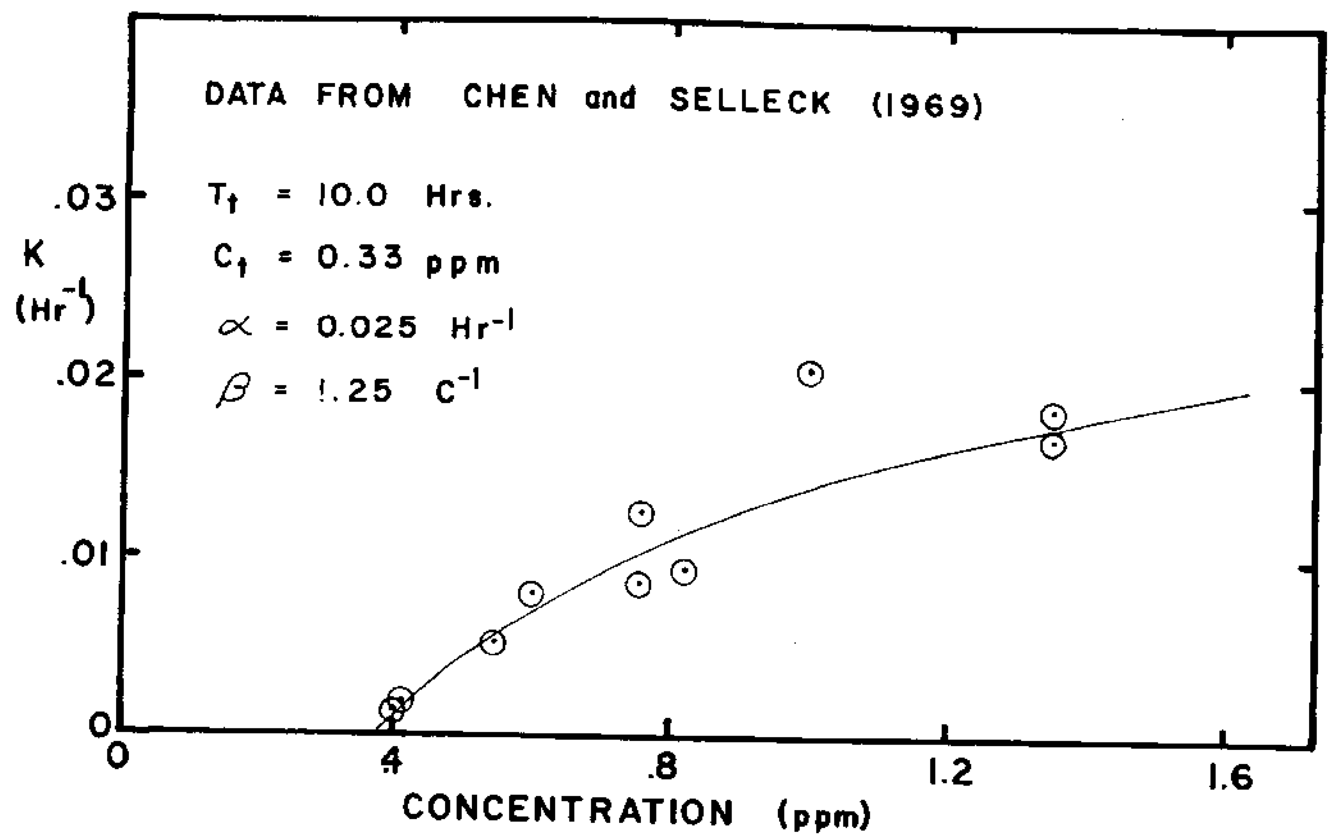


Figure 4.7. Mortality rate coefficient for zinc chloride on guppies.

water is reasonably represented by the maximum concentration in the water source. The dilution capacity of a waterway, as it affects human toxicity, can therefore be evaluated by the amount of spilled pollutant required to produce a given maximum concentration a given distance downstream. This can be evaluated through an algebraic rearrangement of equation (3.8).

A desirable manner to evaluate the dilution capacity of rivers for aquatic toxicity is to compare the amount of a given toxicant that would be required to produce a given percent mortality in the stream a standard distance downstream. In this way, the major parameters -- discharge, velocity and longitudinal dispersion -- interact to produce the concentration distribution to which the aquatic system is exposed.

Percent mortality as a function of a variable concentration of pollutant (variable K) was evaluated by integrating numerically Chick's Law, equation (4.4) over time of passage of the pollutant slug. Time of pollutant passage was determined by distance downstream from the spill site (taken as twenty-five miles), stream velocity, u, and the dispersion coefficient, D. Time of the start of the pollutant slug, t_s, and finish, t_f, were taken at $\pm 2\sigma_x$. From the Gaussian form of equation (3.8), σ_x is given by

$$\sigma_x = \sqrt{2\,DT}\,, \tag{4.6}$$

where T = x/u, is time of arrival of the maximum point of the slug.

Chick's Law was arranged for numerical integration as

$$\Sigma \; \frac{\Delta N}{N} \;=\; \sum_{t_s}^{T_f} -K\Delta T \quad . \tag{4.7}$$

The spill mass (M in eq. 3.8) was then adjusted in successive iterations until the value of K (from eq. 4.5) was such that 50% mortality ($\Delta N/N = 0.5$) was produced during the time of passage of the pollutant slug. This process was performed for each of the toxicant/test organism data sets for which functional values for K were available. Different values for stream velocity and dispersion coefficient were then evaluated.

In order to facilitate these calculations, the initial value for spill size was the amount of spill required to produce the threshold concentration throughout the time of passage of the pollutant slug. An appropriate value for Δt was found to be approximately 300 seconds.

In Figure 4.8, the amount required to produce 50% mortality in a stream with velocity of 1.0 ft/sec and a discharge of 6000 cfs is evaluated at different values of the dispersion coefficient, D. For the toxicants with small T_t, phenol and cyanide, the relation is linear with spill amount required increasing as $D^{\frac{1}{2}}$. The two zinc salts, however, exhibit somewhat different behavior. Where the threshold time, T_t, is three hours, (zinc sulfate) the $D^{\frac{1}{2}}$ slope is only approached at higher values of D. Where T_t is 10 hours, this slope is approached only at extremely high D values.

A similar type of situation exists in Figure 4.9 where spill amount required is plotted against mean stream velocity with D equal

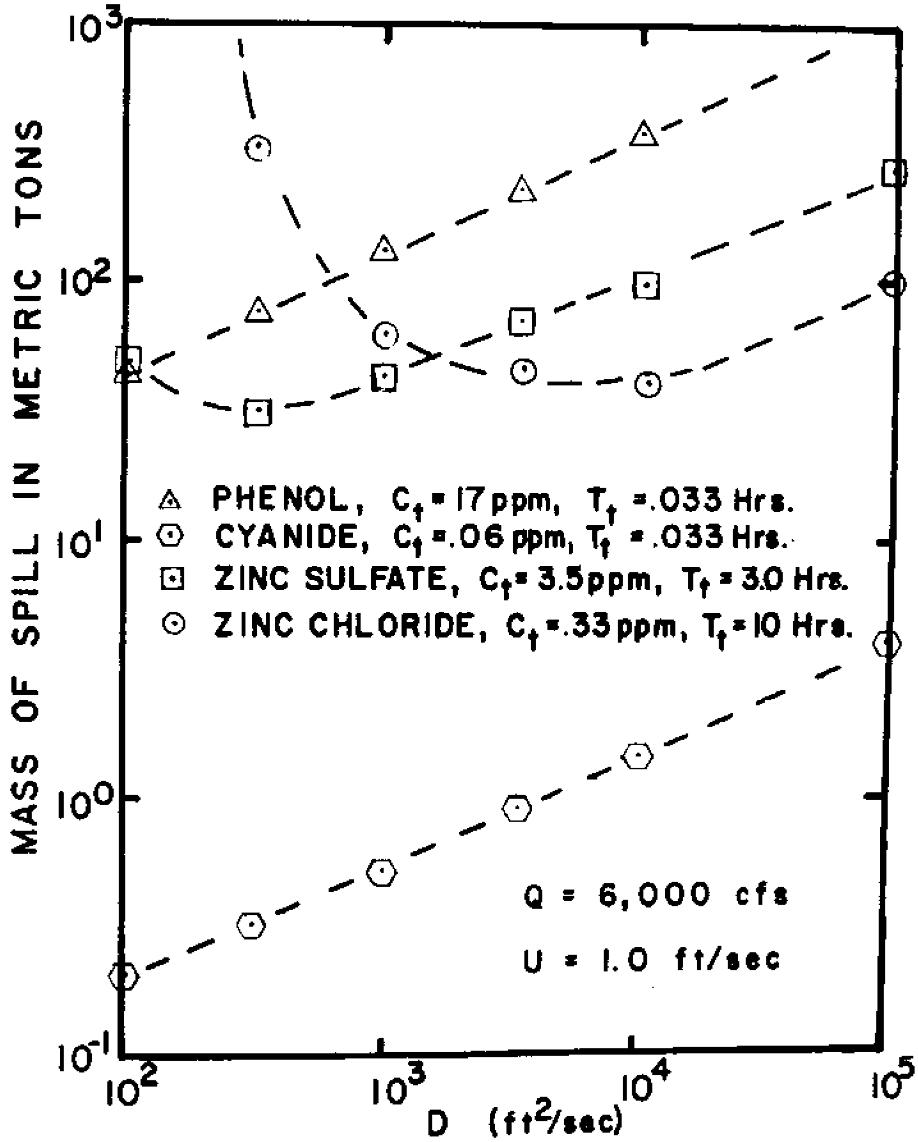


Figure 4.8. Spill size required to produce 50% mortality at different dispersion coefficients.

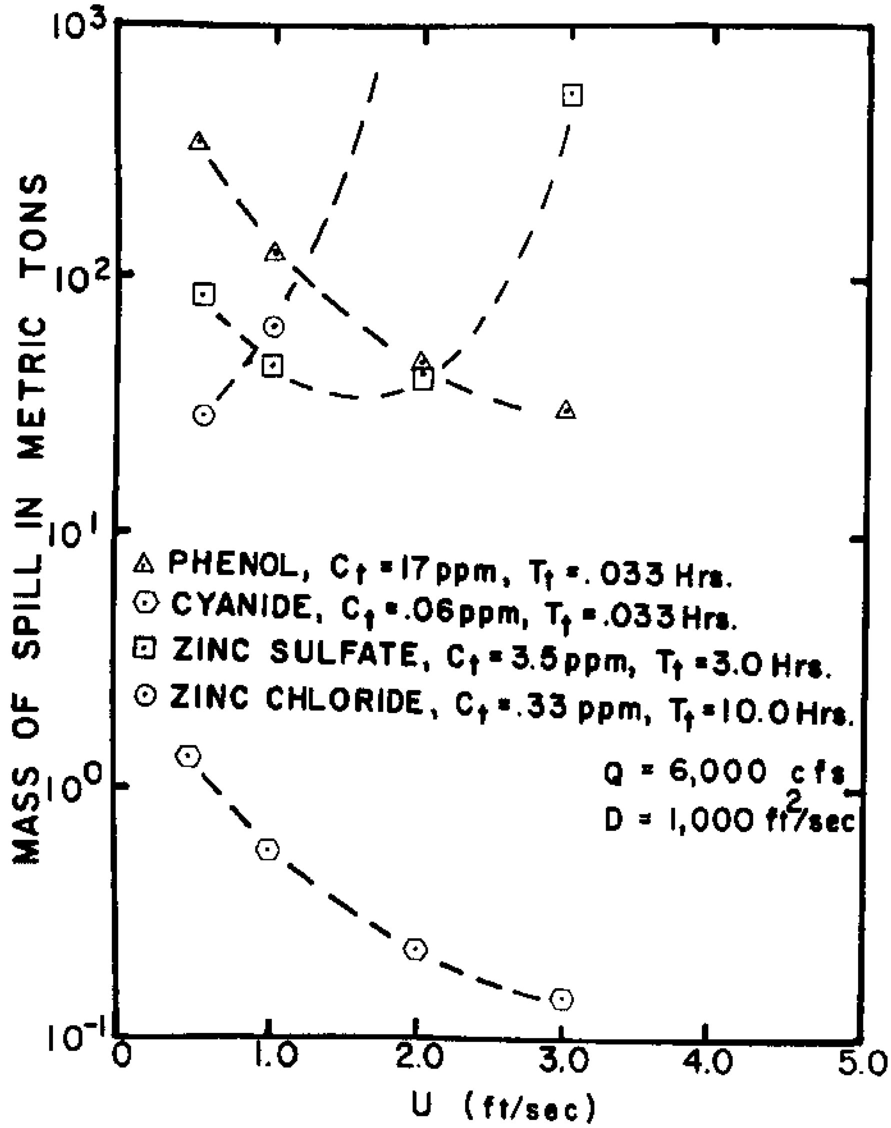


Figure 4.9. Spill size required to produce 50% mortality at different stream velocities.

1000 ft^2/sec. As velocity increases, the time required for the spill to reach the twenty-five miles, and thus the time for the peak concentration to diminish, decreases.

The changed situation with large T_t reflects the requirement that the toxicant must exist at the evaluation point for a time, T_t, before mortality begins to occur. When the variance of the pollutant cloud is reduced, either through small D or large u, more spill is required to produce the necessary duration of exposure.

From the information presented, it appears that T_t is a major determinant of spill impact. This, however, may not be the case. The toxicity information obtained on the two heavy metal salts did not determine if the test organisms would recover if returned to unpolluted water before T_t had elapsed. The question of recovery depends on whether the delayed reaction was due to the time required to induce the pollutant into the organism, or whether the toxin was in the organism and merely slow in acting.

Another factor that reduces the relative importance of T_t variations is that several aquatic toxicologists have reported that the action of many industrial petrochemicals currently shipped in bulk is relatively rapid. Pickering and Henderson (1966) in 96 hour studies on a number of industrial petrochemicals and test organisms, observed few mortalities after the initial 6 to 8 hours. Similar results were reported by Wallen et al.,(1957) in that very small differences were noted between 24-hour and 96-hour LC_{50} values. The ratio of the 6- to the 96-hour LC_{50} concentration for a number of substances reported by Herbert and Shurben (1964) ranged from 1.0 to 0.38, indicating that toxic effects were exhibited early in the test. This relation is not true for all hazardous substances, however. Some materials, generally heavy metals, have a slow toxic action. These are, however, not the major substances carried commercially in the greatest bulk (Dawson, et al., 1970). This information indicates that for the substances of major concern in this study, a short response time is a valid assumption.

One method to evaluate relative dilution capacity of waterway systems is to use the mortality model on each stream. Since the intention of this study is to apply the techniques to a large number of waterways, a method more efficient in time and computation requirements is desired.

The criterion of maximum concentration in the stream at the reference point is desirable because it has already been used as a measure of risk to human populations. If it were found to give results acceptably close to those predicted by the mortality model, it would be a useful method.

Accordingly, the spill mass required for 50% mortality using the model is compared for each toxicant with the spill mass required to produce the threshold concentration, C_t, in the analysis unit. The ratio of the two spill masses versus pollutant cloud variance is presented in Figure 4.10. Pollutant cloud variance from a slug load after 25 miles typically ranges between 5×10^7 and 10^9 ft^2. It can be seen that for the toxicants with short T_t, phenol and cyanide, the

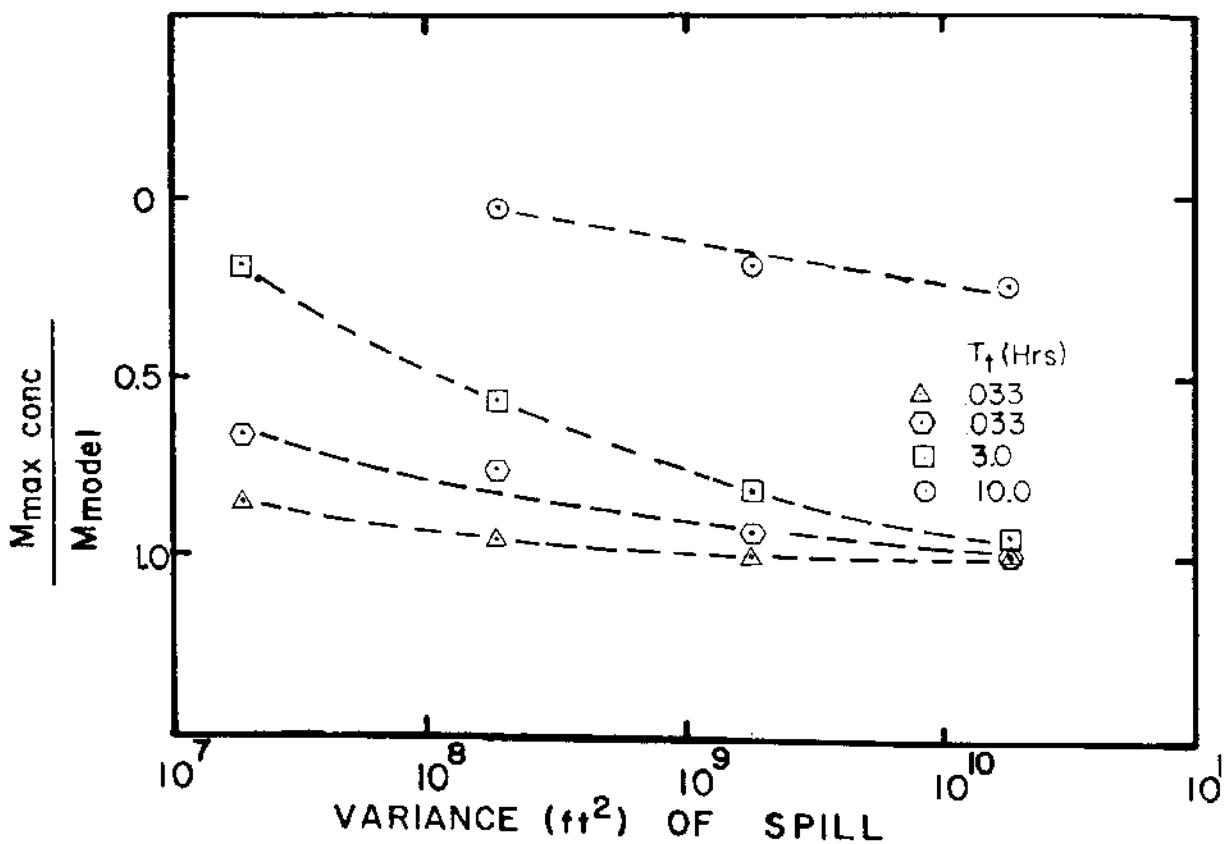


Figure 4.10. The ratio of spill mass required by maximum concentration to that required by the mortality model versus pollutant cloud variance.

difference between the two spill masses is quite small. Where T_t is larger, the mortality model indicates greater spill mass required than would be necessary to produce C_t in the analysis unit.

Where T_t is less than one hour, as available information indicates is true for the majority of the materials shipped in bulk, the use of maximum concentration would produce an underestimate in the spill size required to produce 50% mortality of 20 to 50%.

Another difference between the mortality modeling procedure and application of results to regulatory processes is that the only toxicity information available for most materials is LC_{50} data and not C_t. C_t is the concentration at which no toxicant-induced mortalities will occur if the period of exposure is infinite, while the 96 hour LC_{50} is the concentration at which 50% mortality occurs in the time alloted. The two concentration values are generally quite similar as can be seen from the C_t/LC_{50} from Figures 4.4 through 4.7 respectively of 17/17+, .06/.07, 3.5/4.0, and .33/.56 ppm. When these differences are compared with the range in reported 96 hour LC_{50} values, they are insignificant. For example, the 96-hour LC_{50} for phenol ranges from 16 to 56 ppm (Clemens and Sneed, 1959; Wallen, et al.1957) while even the same worker using different test organisms found the values to range from 24 to 39 ppm (Pickering and Henderson, 1966). If 48 hour LC_{50} values are considered, the range of reported values increases to 5 to 500 ppm (Brown, et al.1957; Portmann and Wilson, 1971).

The error associated with using median toxic concentration information in place of C_t is therefore small compared with the lack of precision in the toxicity information itself. This error will cause an overestimate of the amount of spill required to produce 50% mortality. This overestimate is of the same order and of opposite sign to the error resulting from the use maximum concentration in the analysis unit in place of the mortality model.

It must be cautioned that the fish mortality model does not include many parameters such as temperature, water hardness, pH or sublethal effects on organisms which will affect the impact of a given spill. It is used as a tool to quantify differences in water pollution impact resulting from hydraulic properties and to provide a first approximation to spill size required for a given impact.

The conclusion of this analysis is that the use of maximum concentration at the end of 25 miles together with LC_{50} data is a valid method of characterizing the response of an aquatic system to a spill. The errors associated with the stated simplifications are considerably less than the precision of available toxicity data.

CHAPTER V

DEVELOPMENT OF ANALYTICAL PROCEDURES FOR WATER POLLUTION RISK MANAGEMENT

As discussed earlier, risk management is gaining acceptance as a tool to manage the dangers resulting from the carriage of OHM. For example, the methods developed by Holmes and Narver for the Department of Defense have been used to minimize the risk (defined as the number of deaths per trip) to innocent bystanders resulting from the transportation of poison gas (Selman and Selman, 1974).

Management of water pollution risks from the transportation of OHM involves developing techniques to estimate both the probability of OHM release and the severity of water pollution impact. Probability of release can be and is being estimated from accident frequency analysis (ORI, 1973). Although numerous case studies of specific spills have been conducted, little effort has been directed toward quantifying all the major parameters which affect the impact of a spill. This research develops a procedure whereby the major factors affecting the water pollution impact of a spill can be quantified and used in a risk reduction program. Expenditures for more effective (and expensive) spill control techniques can be concentrated where they will be most beneficial.

In developing a water pollution effect evaluation system, the first decision that must be made is that of the appropriate units. Scales such as number of fish killed or the degree of public outcry were quickly rejected as being too difficult to measure or too subjective. Some progress has been made toward assigning dollar values to water resources, but an element of subjectivity, based on the types and values of uses considered, remains. It appears that at this time the only reasonable way to quantify probable water pollution effects is by using a relative (dimensionless) scale.

The complexity of the water pollution effect problem may be appreciated by the partial list of factors given in Figure 5.1. The factors may be considered as chemical and physical properties of the spilled material, physical properties of the method of release, physical and chemical properties of the receiving waterway, and the biological properties of the exposed organisms.

The properties of OHM as they affect the acute toxic impact on aquatic and marine life have received a great deal of study. Methods have been developed to rate the water pollution hazard of OHM based on the acute toxicity (LC_{50} test data) of the substance. The NAS hazard rating system (NAS-NRC, 1974), described in more detail in the introduction, rates substances on a relative scale (0-4) on the order of magnitude differences in the 96 hour LC_{50} concentration on finfish. The ratings are adjusted for factors which modify the water pollution impact of a spill such as low solubility or high volatility.

The biological properties of receiving waterways, as they affect vulnerability to spills, are extremely complex. Variables include the relative susceptibilities of different organisms to different

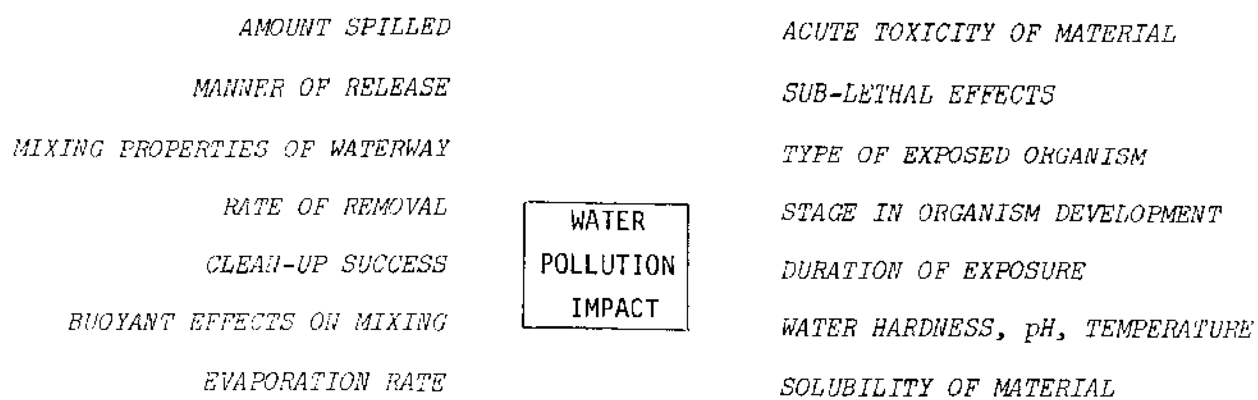


Figure 5.1. Factors affecting water pollution impact of a spill.

pollutants, time of the year, and second order effects through the food chain. The information required to use relative biological vulnerability as an indicator of spill impact on a national basis is not now available. Biological vulnerability may be useful, however, as a risk management tool in specific areas where, for example, a spill during the spawning season for a given organism may have a much greater impact than at other times of the year.

The other major areas which affect water pollution impact of a spill are the physical release, mixing and dilution process in the waterway. The most important factor is the total amount spilled compared to the dilution water available.

The rate at which a pollutant is introduced into the aquatic system is also extremely important. If the release is of long duration, it may be analyzed as a steady state discharge. Where this is the case, the effects of dispersion may be effectively neglected as there will be no significant longitudinal gradients of pollutant concentration.

On the other hand, where the spill is of short duration, such as would occur when a large opening in a tank is caused by heavy impact, the rate of dispersion of the pollutant slug and the stream velocity become the dominant factors. These parameters are important not only because this information is needed to have ample time to shut off water supply inlets, but also because velocity, along with dispersion, determine duration of exposure to the pollutant at a downstream point.

Oil and hazardous material spills are typically of short duration. The rate at which a pollutant slug disperses in a waterway can be critical to the aquatic population. Thomann (1973) notes that dispersion cannot be ignored in large streams if the discharge is at all time variant, a situation which certainly applies to a short duration spill. As can be seen from the analytical solution to the one-dimensional dispersion equation (3.8), the rate of decrease of maximum concentration is proportional to $D^{1/2}$. Measured values of D in natural streams have ranged over two orders of magnitude (Fischer, 1973), indicating that the range of effects attributable to dispersion would be on the order of a factor of ten.

The stream velocity also plays a part in determining effects on aquatic life. A rapidly flowing stream may minimize the exposure time to the toxicant, reducing effects. On the other hand, by reducing the time for the toxicant slug to arrive at a given point, there will be less time for dispersion to dilute the toxicant, thus increasing concentration and effects. The effects of stream velocity on overall waterway dilution capacity are therefore dependent on dispersion and the toxicant/organism considered.

Another important factor in constricted water spill analysis is the achievement of complete cross-sectional mixing. Many finfish have demonstrated an ability to avoid stress from some pollutants (Jones, 1947; 1964; Summerfelt and Lewis, 1967; Wells, 1915). Where relatively uncontaminated water is available, it could be assumed that a significant percentage of the highly motile aquatic community may avoid acute effects. When the entire cross-section is exposed to the pollutant, there is no possiblity of avoidance.

Response of the organisms is another important parameter in dilution capacity analysis. This includes factors such as delayed reactions, destruction of habitats, etc., as discussed in Chapter IV. Biological effects are, however, generally related to the concentration distribution to which the organisms are exposed. As discussed in Chapter IV, the threat to personnel and to aquatic life from the industrial chemicals carried in greatest bulk can be represented by the maximum concentration of the material, provided the point of interest is sufficiently removed from the spill site.

Selection of Quantification Procedures for Dilution Capacity

The two main requirements for a dilution capacity quantification system are analytical accuracy and functionality. These two requirements are to some extent mutually exclusive. In order to be absolutely accurate, a level of information is required that is far beyond present information capabilities. In order to be functional, the quantification system must be as simple and as easy to use as possible. The second requirement dictates a level of analysis free from local time dependent inputs.

After examination of a number of alternatives, the procedures which showed the best promise were to quantify spill mixing and dilution in terms of the dilution volume available (plug-flow analysis) and in terms of the amount of spill released instantaneously (1-D model), required to produce a specified maximum concentration at a standard twenty-five mile distance. Spill dilution capacity ratings are then determined from the two systems. When the spill dilution capacity ratings are combined with the chemical hazard ratings of NAS, an improved (by approximately three orders of magnitude) measure of water pollution impact is achieved.

Decisions on the degree of safety required for appropriate protection of water resources can then be based on this relative indicator of spill impact, combined with a knowledge of the probability of accidental release. The management of the dangers to national water resources from the carriage of OHM may then reflect more closely the actual risks to which water resources are exposed.

Stage I Analysis

The plug-flow model is the basis for Stage I analysis. This model, pictured in Figure 5.2, can be seen to be reasonably representative provided the duration of release is a significant percentage of the time under consideration. In Figure 5.2, the duration of release is 40% of the evaluation period, and the dispersion coefficient used was 1,000 ft^2/sec. Where the duration of release is short or the period of interest is long, the effects of dispersion must be considered.

Available dilution water can be considered in the same fashion as toxicity of a substance because the relation between amount released, dilution available and toxic nature of a substance is linear. Procedures for quantifying relative toxicity of OHM have already

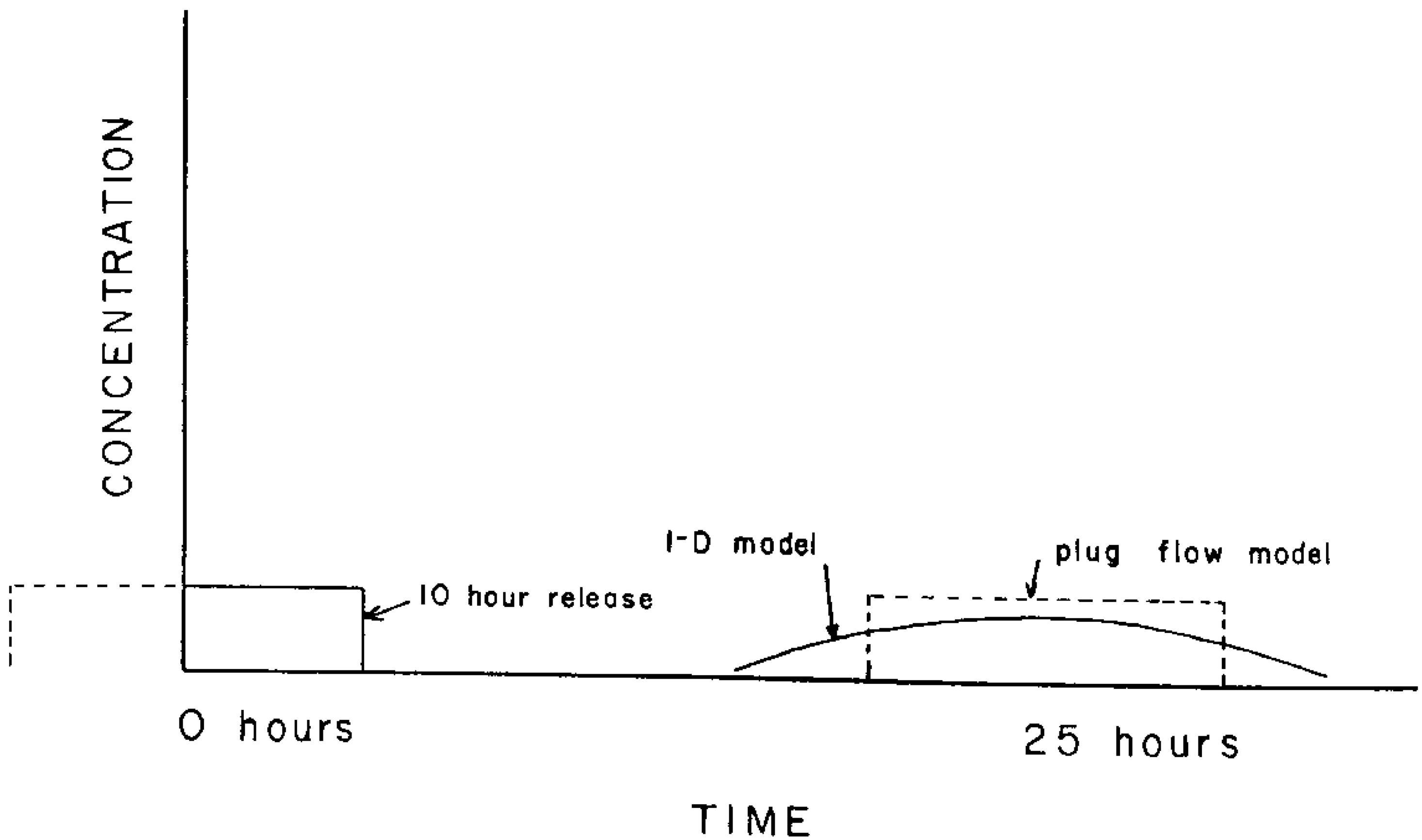


Figure 5.2. Plug flow model for long period release.

been developed by the National Academy of Sciences Committee on Hazardous Materials (1974). As discussed previously, this procedure is a relative scale (0-4) based on order of magnitude differences in toxicity with a class 4 substance 10^4 times more toxic than a class 0. In a similar manner, the expected concentration resulting from a given spill will be 10^2 times as great in an area with a discharge of 1,000 cfs as it would if the discharge were 100,000 cfs.

For this reason, dilution capacity ratings, based on most probable discharge past a specific point, are defined in Table 5.1. Each major waterway is assigned a dilution capacity rating which is a relative measure of vulnerability to OHM spills.

As with assignment of toxic hazard ratings, adjustments must be made to account for departures from the ideal nature of a system. For example, in the NAS aquatic toxicity system, a rating would be lowered from its reported data value if the substance were insoluble to the extent that under natural conditions, it would not harm aquatic life. A similar situation exists with dilution capacity ratings. In estuarine areas, discharge changes very quickly with distance away from tidal influence. In areas where fresh water flow is small, tide-induced velocity drops to near zero in the smaller, upper reaches of an estuary, while discharge past the mouth is quite large. A range of closely spaced dilution capacity ratings would be difficult for the shipper and regulatory agency to use. For this reason, dilution capacity ratings were averaged over workable geographic

TABLE 5.1. Stage I Waterway Dilution Capacity Rating Scale.

Class	Median Discharge ft^3/sec	m^3/sec
0	> 100,000	> 2,832
1	100,000-10,000	2,832-283
2	10,000-1,000	283-28.3
3	1,000-100	28.3-2.8
4	100 <	2.8 <

units. The rationale for these adjustments is presented along with the data in Appendix A.

Stage II Analysis

The critical concept in the analysis of OHM water pollution problems is the relation between toxicity of the material, volume of the material released, and the dilution capacity of the system. While a Stage I type analysis is useful and a decided improvement over consideration of material toxicity alone, more improvement is possible through consideration of spill size, hydraulic characteristics and probable dispersion in the waterway.

The method selected to quantify the effects of spill size and pollutant mixing is to determine for each waterway, the amount of release required to produce a specified severe environmental impact. This impact was considered to have occurred when a concentration of 1,000 ppm (NAS Class 0 toxicant LC_{50} >1,000 ppm) existed in the analysis unit.

In non-tidal systems, the procedure is to solve the analytical solution to the one-dimensional dispersion equation (eq. 3.8) for the amount of spill required to produce the critical concentration twenty-five miles downstream. Inputs to eq. (3.8) are discharge velocity and the longitudinal dispersion coefficient. Methods for obtaining this information are given in the next chapter.

The 1-D model is the opposite of the plug-flow model in that the 1-D model applies to an instantaneous rather than a continuous release. The maximum concentration, and as indicated in Chapter IV, the greatest effect on aquatic life, occur with an instantaneous release. This situation is illustrated in Figure 5.3. An actual spill situation will produce a concentration distribution between that predicted by the plug flow and the 1-D models.

Twenty-five miles was selected as the unit of analysis because it is of sufficient distance to insure complete cross-sectional mixing (and thus the applicability of the 1-D model) in all but the largest rivers (Fischer, 1967; Stewart, 1967). At the same time, twenty-five miles is a small enough analysis unit so that major changes in the waterway do not make the assumptions of constant area and velocity invalid. In addition, a spill which damages aquatic life for a twenty-five mile stretch is a reasonable definition of a significant environmental impact.

Since tidal systems are often not dominated by advective flow, the concept of a specified area subjected to serious environmental impact needs modification. In the absence of large freshwater flows, tidal action produces flows which move a parcel of water back and forth past a point. The length of this motion is called the tidal excursion. Where significant freshwater flows exist, the length of the ebb excursion is generally greater than the flood. The volume of the excursion may usually be approximated by the surface area above a given point multiplied by the change in tide height, or tidal prism. The tidal prism, pictured in Figure 5.4, is the volume that is exchanged with the ocean during each tidal cycle.

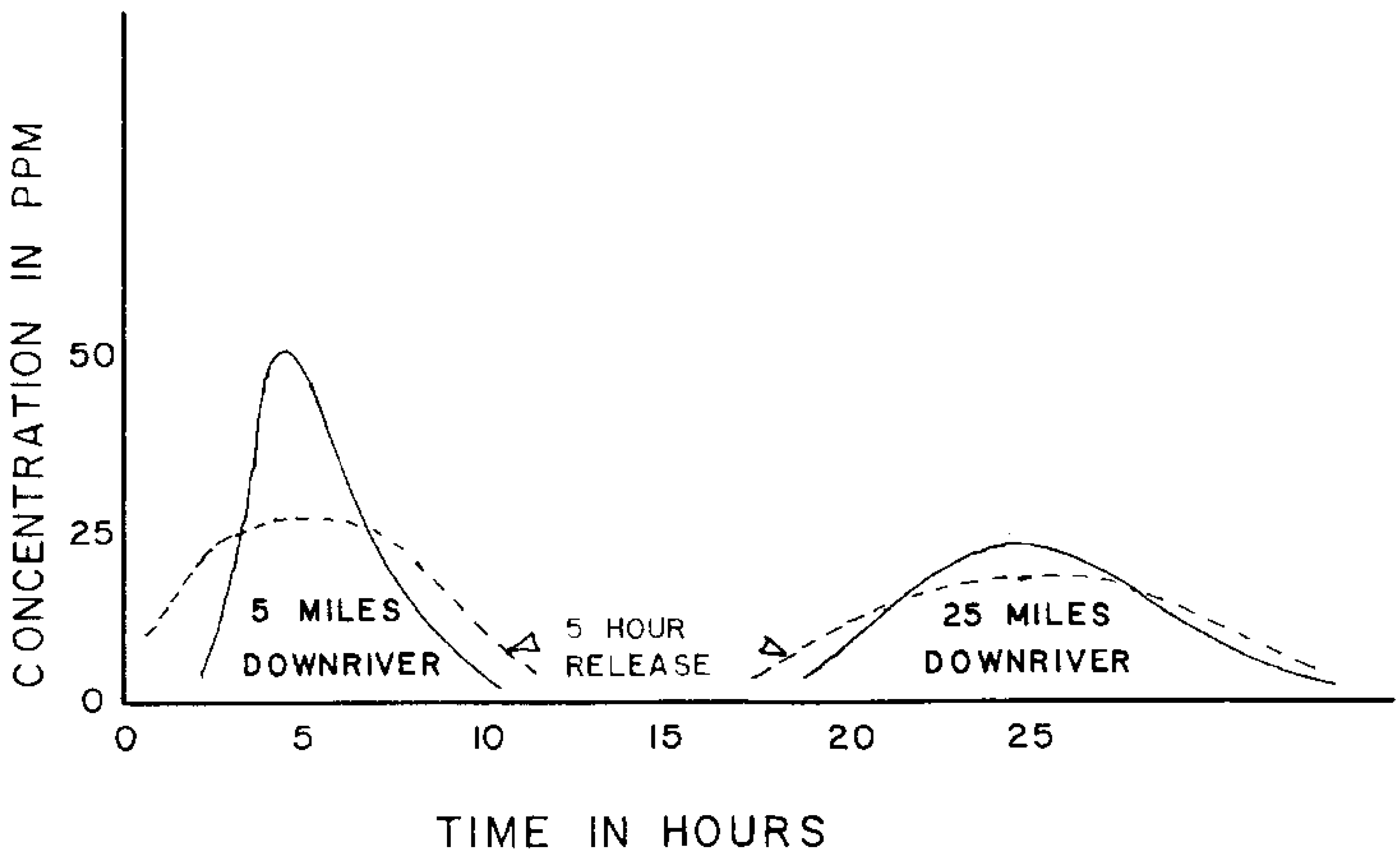


Figure 5.3. Dispersion of instantaneous pollutant release.

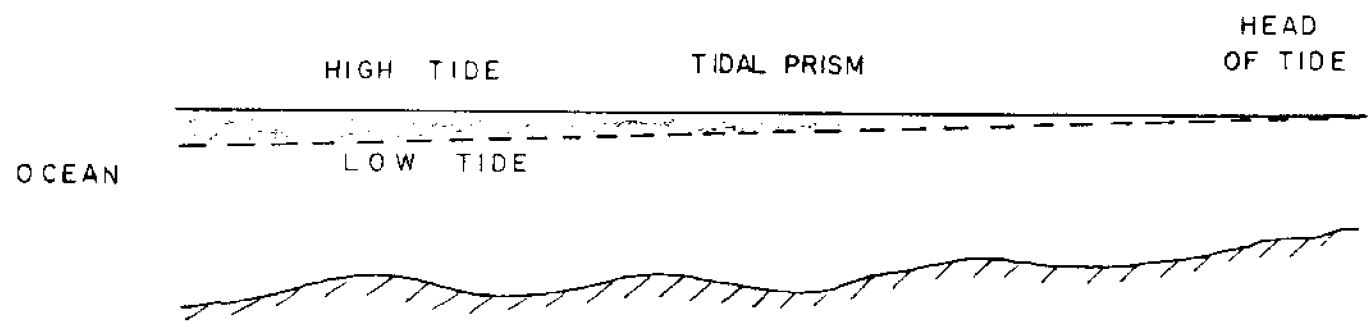


Figure 5.4. Tidal prism.

The tidal excursion was selected as the most appropriate analysis unit because it is an approximate upper bound on the volume that is initially exposed to the spilled pollutant. If the spill were to continue for several tidal cycles, approximately the same parcel of water will receive the pollutant. With time, advection from freshwater flows and dispersion will increase the area exposed and reduce the concentration, but the process of removal of the spill from the exposed area will be somewhat slower than in a one-dimensional non-tidal system.

The analysis procedure for tidal systems was to determine the amount of spilled material that would be required to produce the critical concentration throughout the tidal excursion. Advection and dispersion were not considered in the tidal analysis because advective motions are often small and difficult to predict accurately and the state of art of estuarine dispersion coefficient prediction is not as well developed as in non-tidal areas.

The spill mass necessary to produce 1,000 ppm in the analysis unit was then converted to a relative rating similar to the Stage I system. The scale defining this rating is given in Table 5.2. In addition, resolution of these ratings is increased by the inclusion of one decimal, calculated by

$$\text{Rating} = \text{Log}_{10} \left(\frac{10^6}{\text{tons}}\right) . \tag{5.1}$$

The advantage of this representation is that dilution capacity which may be very similar but in different classes will not appear as an order of magnitude difference.

TABLE 5.2. Stage II Waterway Dilution Capacity Rating Scale.

Class	Spill Quantity (Metric Tons)
0	$>$100,000
1	100,000-10,000
2	10,000-1,000
3	1,000-100
4	100$<$

Use of Stage II Analysis

The relationship between spill size and distance exposed to a specified concentration is illustrated in Figure 5.5. This figure was constructed using the one-dimensional dispersion model, equation (3.8). It can be seen that if the spill volume were reduced by one-half, the distance exposed to the critical maximum concentration would be reduced to one-fourth the original distance. Similarly, the analysis in Chapter IV indicates that for distances sufficiently removed from the spill site, the same size spill of a substance which produces a similar toxic impact at one-half the concentration will expose four times the original stream distance.

With the system response predicted by Figure 5.5 and the spill size that would theoretically produce an impact area of twenty-five miles, environmental planners have the tools to estimate areas at risk from a given size and toxicity shipment. For example, with the assumptions of a conservative material, if a 4,800-ton spill were required to produce 1,000 ppm twenty-five miles downstream at mile 124 of the Monogahela River, only 48 tons of material with a threshold concentration of 10 ppm would be required to produce the same impact area. If 100 tons were spilled, the impact area from Figure 5.5 would be four times the original or 100 miles, nearly the entire navigable length of the river!

This example does not allow for changes in the river, such as increased discharge downstream of the spill site, or the removal of the toxicant by evaporation, sorption or decay. It does demonstrate

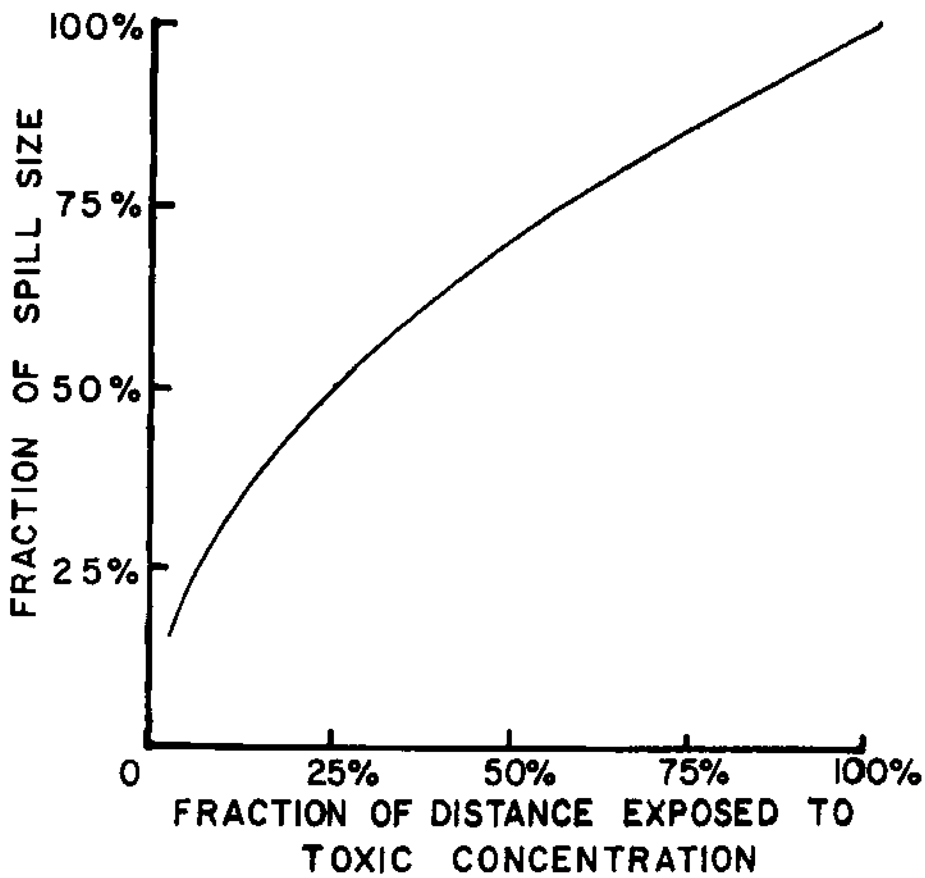


Figure 5.5. Relation of spill size to distance exposed to toxic concentration.

how areas at risk can be estimated so that appropriate safety precautions can be taken. Examples of the possible uses of the dilution capacity ratings are given in Appendices B and C.

CHAPTER VI

APPLICATION OF ANALYTICAL PROCEDURES TO MAJOR INLAND AND INTRACOASTAL WATERWAYS

The following procedures were utilized to collect and analyze the data required to conduct the dilution capacity analysis as described in the previous chapter.

The procedures should ideally be applied to every waterway; however, limitations in available resources restricted the study to the great majority of the major inland and intracoastal waterways. The waterways analyzed are pictured in Figure 6.1 and listed in the tables to follow.

Stage I - Non-Tidal Systems

The available dilution water in a river varies greatly with time. Stream flow records for most waterways demonstrate that the average discharge is not the discharge that would most likely be encountered. This is because flood flows are frequently a factor of ten greater than average flows. For example, the average of eleven periods, each with a flow of 1000 cfs and one period where the flow is 10,000 cfs would be 1750 cfs. Yet the most likely flow to be encountered would be 1000 cfs.

Hydrologists have met this problem by using a flow duration

Figure 6.1. Waterways evaluated in this study.

curve obtained by plotting flow magnitudes versus the percent time that flow was exceeded. This flow duration curve can be used to determine the flow that would be exceeded a given percentage of the time. An example of a typical flow duration curve plotted with linear coordinates before and after flow regulation is shown in Figure 6.2. It can be seen that flow regulation significantly alters the shape of the flow duration curve.

Flow duration curve data were obtained from the U.S. Geological Survey (USGS), the organization charged with the collection and reporting of streamflow data. These data were available for most stations from the various USGS district offices in the form of computer outputs giving the percentage of time a given discharge was exceeded. An example of one of these computer outputs is presented as Figure 6.3.

Streamflow was evaluated on the basis of the 50% flow duration--that is, the discharge at which half the time the flow will be greater and half the time the flow will be less. This flow was obtained from the duration tables by linear interpolation between adjacent flow percentages. The discharge exceeded 90% of the time was also determined in the same manner, in order to identify rivers where significant differences in dilution capacity may exist during low flow periods. Discharge was interpolated linearly between gaging stations.

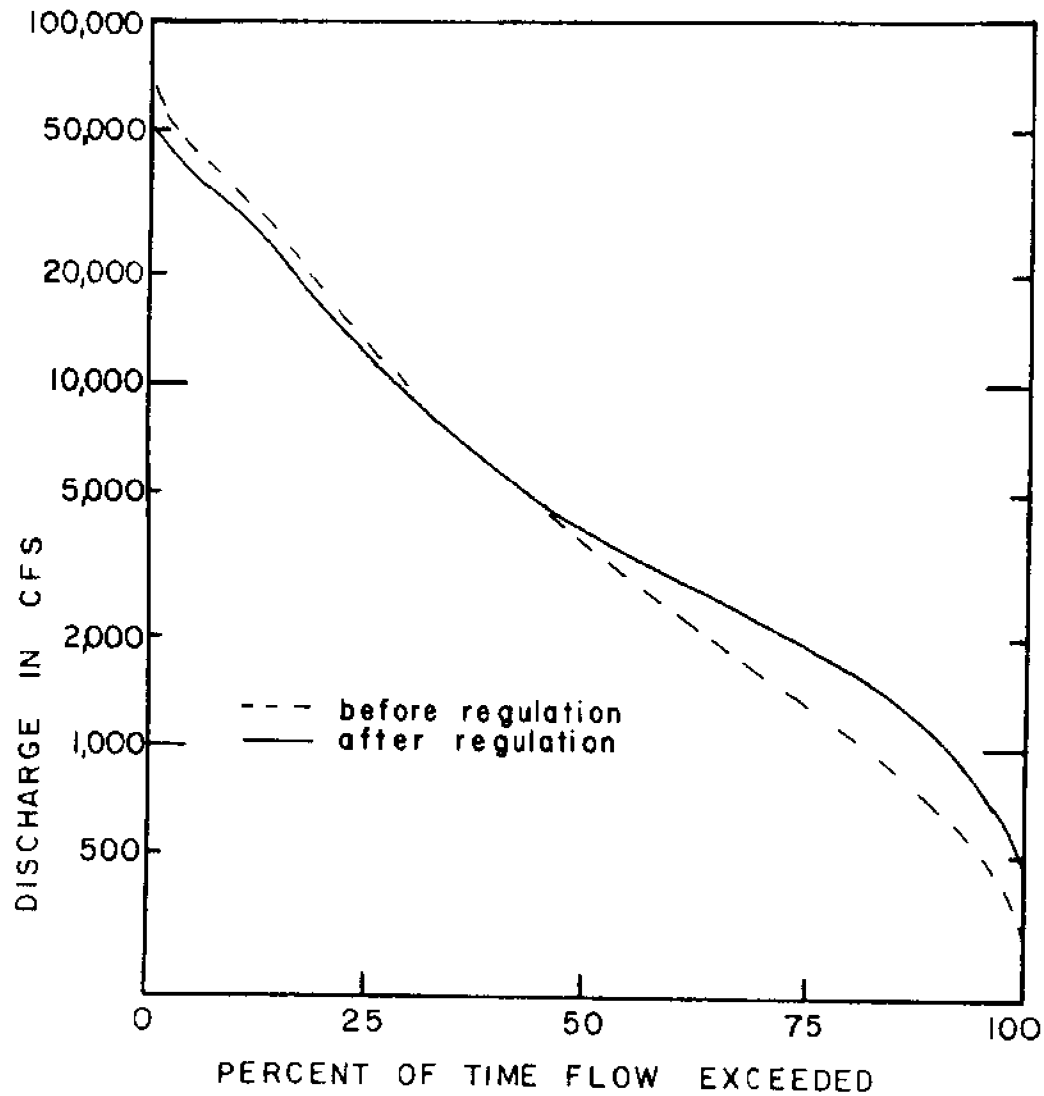


Figure 6.2. Flow duration curve for the Green River at Calhoun, KY, before and after flow regulation.

STATION_NUMBER 1-3185.00

DURATION TABLE OF DAILY DISCHARGE

HUDSON RIVER AT HADLEY, N. Y.

NUMBER OF DAYS IN CLASS

YEAR \ CL	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	CFS_DAYS
1941			1	23	19	21	14	17	18	19	43	30	40	42	22	13	9	3	4	1	3	5	2	3	3	2	4	2	2							694157.0
1942			1	3		3	1	9	22	46	58	40	16	28	24	10	16	13	10	8	19	5	12	8	1	4	4	2	2							939390.0
1943										11	21	18	31	57	45	32	32	24	19	14	12	8	3	7	7	4	10	7	3							1239410.0
1944				1		1	9	11	22	64	69	37	30	15	14	8	14	12	9	3	4	2	5	7	3	8	7	3	5	3						968526.0
1945						2		12	23	44	79	26	16	17	18	14	17	7	8	8	15	8	14	9	4	3	11	6	2	2						1117430.0
1946							4	5	6	18	39	16	10	24	39	30	32	30	25	16	13	18	6	14	7	4	7	1	1							1205662.0
1947										10	31	21	24	53	29	20	23	26	23	13	15	4	11	10	15	9	11	5	8		3		1			1515650.0
1948						11	22	26	26	49	56	41	17	14	11	10	14	11	4	4	14	8	8	2	2	1	3	5	3	4						691894.0
1949		1	7	3	12	29	14	18	22	13	32	13	11	16	31	25	23	16	11	11	12	14	15	2	4	2	4		1			1	1		1	950957.0
1950									21	39	39	37	26	34	22	23	20	18	14	14	8	14	8	7	8	4	4	1	2							1036805.0
1951								1	29	16	13	18	25	71	31	27	33	28	17	4	7	2	9	7	4	5	6	2	6	3	1					1179434.0
1952						2		2	27	24	22	27	15	26	15	30	36	29	22	10	12	11	17	7	8	1	3	5	9	4	2					1360136.0
1953					7	12	12	32	45	31	18	20	15	18	25	15	13	11	10	14	12	11	9	7	8	4	4	4	1	2	3		2			1106958.0
1954							9	13	39	40	26	24	36	22	13	13	9	14	22	9	11	11	8	8	11	12	4	3	4	1	1					1166123.0
1955						4	27	17	22	11	22	41	24	36	24	18	17	18	23	9	12	7	9	2	1	7	2	4	6	2						1052471.0
1956								5	22	53	96	29	18	14	9	14	14	12	13	5	11	16	11	9	2	4	5	1	3							961806.0
1957				1	6	12	15	16	15	11	45	31	43	33	27	20	21	24	18	11	7	4	4	1												734155.0
1958					6	8	13	10	20	35	34	22	53	24	26	18	16	14	6	5	13	7	9	5	4	3	2	2	4	4	1	1				1026547.0
1959					4	2	1		17	32	57	47	43	44	19	14	16	14	17	4	5	2	2	4	1	1	11	6	3							945852.0
1960							2	13	9	13	25	19	20	40	36	22	21	19	19	16	23	11	10	9	10	4	8	7	6	3	1					1400274.0
1961					1	11	39	24	22	44	46	27	17	18	12	13	14	17	16	10	12	5	5	3	4	2	2	1								764400.0
1962		3	3	4	3	2	3	31	26	73	92	22	19	15	6	7	3	6	3	4	4	4	3	4	8	4	6	3	2	2						602227.0
1963						1	6	21	28	41	61	35	32	31	13	10	5	11	11	15	13	4	7	6	3	5	2	1	2	1						876519.0
1964			1		10	44	43	46	25	34	28	17	8	9	7	13	17	10	8	8	12	7	3	7	2	2	2	1	2							710596.0
1965		1	8	14	26	30	45	36	39	39	33	18	13	17	11	2		3	4	1	2	6	10	5		2										513800.0
1966					3	7	20	19	16	14	39	38	21	29	19	21	30	13	20	17	15	6	3	6	7	2										880045.0
1967							2	20	44	43	67	56	29	19	13	5	6	7	8	6	7	6	7	4	5	4	4	2	1							796908.0

CLASS	CFS	TOTAL	ACCUM	PERCT	CLASS	CFS	TOTAL	ACCUM	PERCT	CLASS	CFS	TOTAL	ACCUM	PERCT	CLASS	CFS	TOTAL	ACCUM	PERCT
0	0.00	0	16801	100.0	9	940.00	1229	14095	83.9	18	3500.0	660	3665	21.8	27	13000	137	333	1.9
1	292.00	20	16801	100.0	10	1100.00	1856	12866	76.6	19	4100.0	457	3005	17.9	28	15000	104	156	1.1
2	340.00	81	16781	99.9	11	1300.00	1401	11010	65.5	20	4700.0	523	2548	15.2	29	16000	44	92	.5
3	390.00	148	16700	99.4	12	1500.00	1243	9609	57.2	21	5500.0	379	2025	12.1	30	20000	30	48	.2
4	450.00	171	16552	98.5	13	1700.00	1372	8366	49.8	22	6300.0	354	1646	9.8	31	24000	6	18	.1
5	520.00	331	16381	97.5	14	2000.00	969	6994	41.6	23	7300.0	315	1292	7.7	32	27000	9	12	.0
6	610.00	431	16050	95.5	15	2300.00	767	6025	35.9	24	8500.0	251	977	5.8	33	32000	2	3	.0
7	700.00	635	15619	93.0	16	2600.00	851	5258	31.3	25	9800.0	181	726	4.3	34	37000	1	1	
8	810.00	869	14984	89.2	17	3000.00	742	4407	26.2	26	11000.0	212	545	3.2					

Figure 6.3 Flow duration table for the Hudson River at Hadley, N.Y.

Stage I - Tidal Systems

Available dilution water moving past a point was estimated in two ways, depending on data availability. The preferred data source was tidal current information presented in the Tidal Current Tables of the National Ocean Survey (1972). On locations where these data were not available, such as long intracoastal waterway reaches, the method of tidal cubature was used to estimate discharges.

Information in the Tidal Current Tables is presented as maximum velocity measurements at a station, for flood and ebb currents averaged over all strengths of tide. The average current velocity was obtained by making the assumption that the tidal current velocity was a sinusoidal function, and integrating over one half tidal cycle. The result is that average velocity, $U_{ave} = 2/\pi\ U_{max}$, where U_{max} is the average of the ebb and flood maximum velocities given in the Tidal Current Tables.

The maximum strength of the tide is a very complex, though periodic, function of time. It was assumed that this function was not skewed to an appreciable extent toward high or low current velocities so that the average maximum velocities given by the Tidal Current Tables, converted to average velocities, do represent a valid estimate of median current velocity.

This estimated median tidal current velocity was converted to mean channel velocity, Q/A, by multiplying an empirically determined coefficient, 0.75. This coefficient is the ratio of mean channel velocity to maximum velocity pictured in Figure 6.4 for several large rivers and estuaries (USGS data). Cross-sectional areas were obtained from navigation charts with soundings corrected for mean tidal height.

In many smaller estuarine areas such as Intracoastal Waterway sections, there is no tidal current information published by the National Ocean Survey. For these areas, discharge estimates were obtained by the method of tidal cubature. In this procedure, discharge past a point as pictured in Figure 6.5 is related to surface area in the stream, change in elevation with time, and freshwater discharge Q_{fw}

$$Q = A_s \frac{dH}{dT} + Q_{fw} , \qquad (6.1)$$

where A_s is the surface area above a segment of the stream. With H given by a sinusoidal function,

$$H_t = \tfrac{1}{2}(H_{max} + H_{max} \operatorname{Cos}(\frac{2\pi t}{T_p})), \qquad (6.2)$$

where H_{max} is the maximum height of tide above datum, H_t is the tide height at time t, and T_p is the tidal period, the discharge past a point (segment) becomes

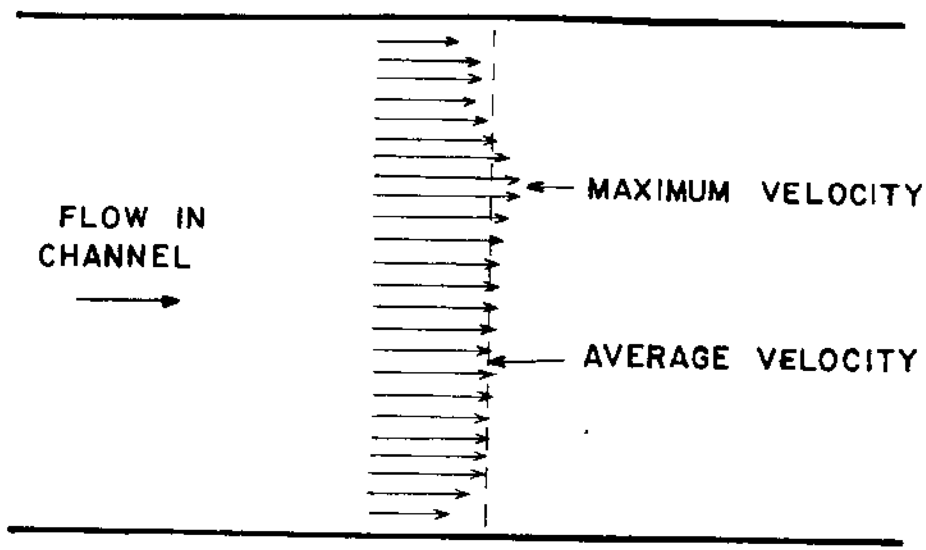


Figure 6.4. Relation of average velocity, Q/A, to maximum velocity in a typical channel.

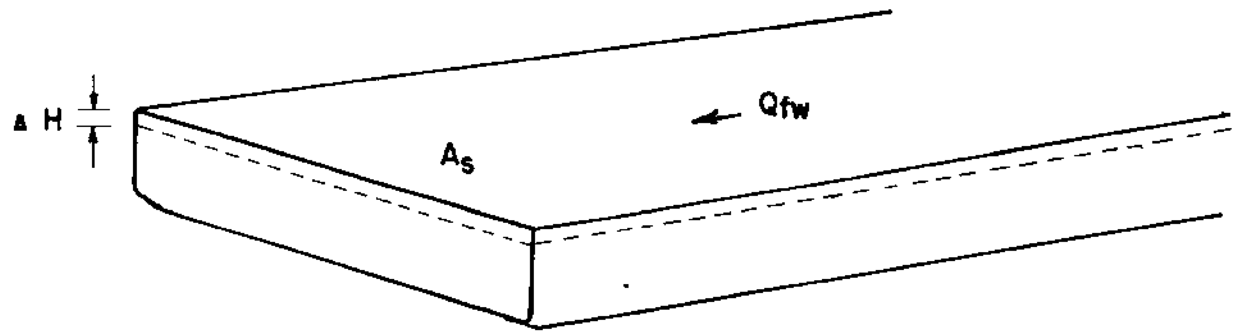


Figure 6.5. Tidal cubature.

$$Q = A_s \frac{H_{max}}{2} (\mathrm{Sin} \ (\frac{2\pi t}{t_p}) \ ) + Q_{fw} \quad . \tag{6.3}$$

The average change in tide height with time is the tidal range divided by one-half the tidal period. Tidal ranges are published in the Tide Tables of the National Ocean Survey (1972) at several locations along most major estuarine systems.

Tidal ranges vary to some extent with position in an estuary. If the estuary narrows sharply from a wide entrance at the ocean, the tidal range at the upper end will frequently be greater than at the lower end. On the other hand, if the width is relatively constant, tidal range will decrease with distance up the estuary (Ippen, 1966). In all calculations of discharge by tidal cubature, the average tidal range for each segment was used.

Where no information was available on tide heights in the upper reaches of a tidal system, e.g. interior locations on the Intracoastal Waterway, the tide range was conservatively taken as constant along its length. Where this is so, discharge as a function of length along a channel may be obtained by:

$$Q = 2 \, W \, L(x) \quad T_r/T_p \quad , \tag{6.4}$$

where W is width, L(x) is length from the ocean, and T_r is tidal range.

The Stage I dilution capacity ratings, based on the scale in Table 5.1 (p. 89) and adjusted as described in Chapter V, are presented in Table 6.1. The data and adjustment rationale, if any, are presented in Appendix A.

TABLE 6.1. Stage I Waterway Dilution Capacity Ratings.

Location	Mile	Rating
Alabama River	278 - 0	1+[a]
Allegheny River	72 - 0	2
Apalachicola River	104 - 0	1
Arkansas River Waterway	488 - 395	2
	395 - 0	1+
Atlantic Intracoastal Waterway	0 - 34	3
	34 - 102	0
	102 - 128	3
	128 - 185	0
	185 - 205	2
	205 - 297	2
	297 - 309	1
	309 - 341	2
	341 - 464	2
	465 - 576	1
	576 - 596	2
	597 - 714	1
	714 - 729	1
	730 - 822	2
	822 - 858	1
	858 - 987	0
	987 - 1013	3
	1013 - 1034	1
	1034 - 1080	3
	1080 - 1089	1
Chattahooche River	155 - 0	2
Black Warrior and Tombigbee River System	466 - 0	2
Columbia River	340 - 0	0
Connecticut River Estuary	45 - 0	1
Cumberland River	385 - 0	1+

[a]The "+" indicates that the 90% flow is in a higher rating class.

TABLE 6.1, continued

Location	Mile	Rating
Delaware River Estuary	132 - 90	1
	90 - 0	0
Flint River	29 - 0	2
Green and Barren		
Rivers	168 - 0	2+
Barren	30 - 0	2+
Gulf Intracoastal Waterway		
West Florida Section		
(miles from San Carlos Bay, FL)	0 - 50	0
	50 - 60	3
	60 - 95	0
Florida Panhandle Section		
(miles from New Orleans, LA)	380 - 350	0
	350 - 335	2
	335 - 313	3
	313 - 273	0
	273 - 254	3
	254 - 167	0
	166 - 160	0
	160 - 150	4
	150 - 113	0
	112 - 37	0
Texas Coast (miles westward		
from New Orleans, LA)	265 - 349	3
	349 - 363	1
	363 - 455	3
	455 - 655	1
Houston Ship Channel	52 - 28	2
(miles from Galveston)	28 - 0	0
Hudson River below Troy, NY	140 - 90	1
	90 - 0	0

TABLE 6.1, continued

Location	Mile	Rating
Illinois Waterway	354 - 0	2
James River Estuary	87 - 0	1
Kanawha River	91 - 0	2
Kennebec River Estuary	36 - 10	1
	10 - 0	0
Kentucky River	259 - 0	2+
Mississippi River (Upper)	857 - 812	2
(miles from Cairo, IL)	812 - 195	1
	195 - 0	0
Mississippi River (Lower)	956 - 0	0
Missouri River	732 - 0	1
Monongahela River	129 - 0	2+
New York State Barge Canal		
Erie Canal		
Tonawanda to Three Rivers		3+
Three Rivers to Troy		2
Champlain Canal and Hudson River		2
Ohio River	981 - 0	1
Okeechobee Waterway	0 - 39	4
(miles from St. Lucie Inlet)	39 - 77	1
	77 - 140	4
Penobscot River Estuary	19 - 0	1
Sacramento River	145 - 0	2
St. Johns River	152 - 100	2+
	100 - 0	1
San Joaquin River	127 - 0	3
Savannah River and Estuary	215 - 205	2
	10 - 0	1

TABLE 6.1, Continued

Location	Mile	Rating
Snake River	140 - 0	1
Tennessee River	652 - 0	1
Willamette River	132 - 0	1

Stage II - Tidal Systems

The amount of spill required to produce 1000 ppm concentration in the tidal excursion volume was calculated at points along each major estuarine system. The procedure followed was to first calculate the volume of the tidal excursion at a point

$$\text{Volume} = Q\ T_p/2, \tag{6.5}$$

and then convert this to the amount of spill in metric tons required:

$$\text{Tons} = \text{Volume} \times 62.4\ \text{lbs/ft}^3 \times 1/2200\ \text{tons/lb} \times .001. \tag{6.6}$$

Stage II - Non-Tidal Systems

The amount of spill required to produce the 1000 ppm critical concentration twenty-five miles downstream from a spill site was evaluated at points along the major inland waterways. Mass of spill in metric tons required was obtained for each waterway reach using an algebraic rearrangement of equation (3.8) (p. 35):

$$\text{Tons} = .001 \times A \times 62.4\ \text{lbs/ft}^3 \times 1/2200\ \text{tons/lb} \times (4\pi DT)^{\frac{1}{2}}, \tag{6.7}$$

where T is the time for the peak of the pollutant slug to reach twenty-five miles, $T = 25\ \text{miles}/u$, A is cross-sectional area in feet squared, and D is the longitudinal dispersion coefficient in feet squared per second.

Data on stream velocity, width, height, and cross-sectional area at gaging stations were obtained from district offices of the U.S. Geological Survey. The primary source of the data was discharge measurement summary sheets (Form 9-207) maintained in the various USGS District Offices. An example is shown in Figure 6.6. These forms give cross-sectional area, gage height, width (and thus mean depth) and mean velocity for the particular discharge. When these parameters are plotted versus discharge, curves are obtained from which the mean velocity, area, gage height and width can be determined for the flow which exists 50 percent of the time. These values were also obtained in a similar fashion for the flows which exist 90 percent of the time in order to examine the effect of system flow variability on the analysis. A plot of Form 9-207 information is presented in Figure 6.7. The flows marked are those exceeded 90 percent and 50 percent of the time.

At the 90 percent and 50 percent low flows, stream velocities are determined primarily by the action of control structures. Where gaging stations are located near control structures, results obtained may not be representative of the entire reach.

The procedure that was evolved to allow for this problem was to obtain navigation charts for the waterway from the U.S. Army Corps of Engineers. The position of the gaging stations relative to control structures was noted on the chart. A determination was then made if this station was representative of the entire reach or was a special case such as a narrow channel immediately downstream

9-207
(Nov. 1957)

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY (WATER RESOURCES DIVISION)

Discharge measurements of Green River at lock 2 at Calhoun Ky

No.	Date	Made by--	Width	Area	Mean velocity	Gage height	Discharge	Rating 13 Shift adj.	Rating 13 Percent diff.	Method	Number meas. sections	Gage height change	Time	Meas. rated
	1965		Feet	Sq ft	Fps	Feet	Cfs	Feet				Feet	Hr	
588	Sept 1	Miles	293	4270	0.44	10.33	1,890		0	.2 8	26	-01	1.33	F
589	Oct 5	Bailey	280	4,480	.77	10.94	3,470		-7.4	.2 8	24	0	1.50	G
590	Nov. 3	Miles	296	4,410	.72	11.04	3,160	-.17	-4.5	.2 8	24	+01	1.08	
591	Dec. 2	do	302	4,550	.82	11.24	3,740	-.20	-2.1	.2 8	25	+02	1.00	G
592	1966 Jan. 6	do	418	8,640	3.53	18.68	*39,100 30,500		-2.2	.2 8	36	+11	2.33	G
593	Feb. 9	do	314	5,860	1.35	12.26	7,910		-4.0	.28	28	+05	1.33	G
594	Mar. 3	do	320	6,300	1.73	12.92	10,900		0	.28	29	0	1.00	G
595	Apr. 6	do	290	4280	.42	10.21	1,780	+05	+2.9	.2 8	25	0	1.25	F

Figure 6.6. U.S. Geological Survey Form 9-207 for the Green River at Lock 2, Calhoun, KY.

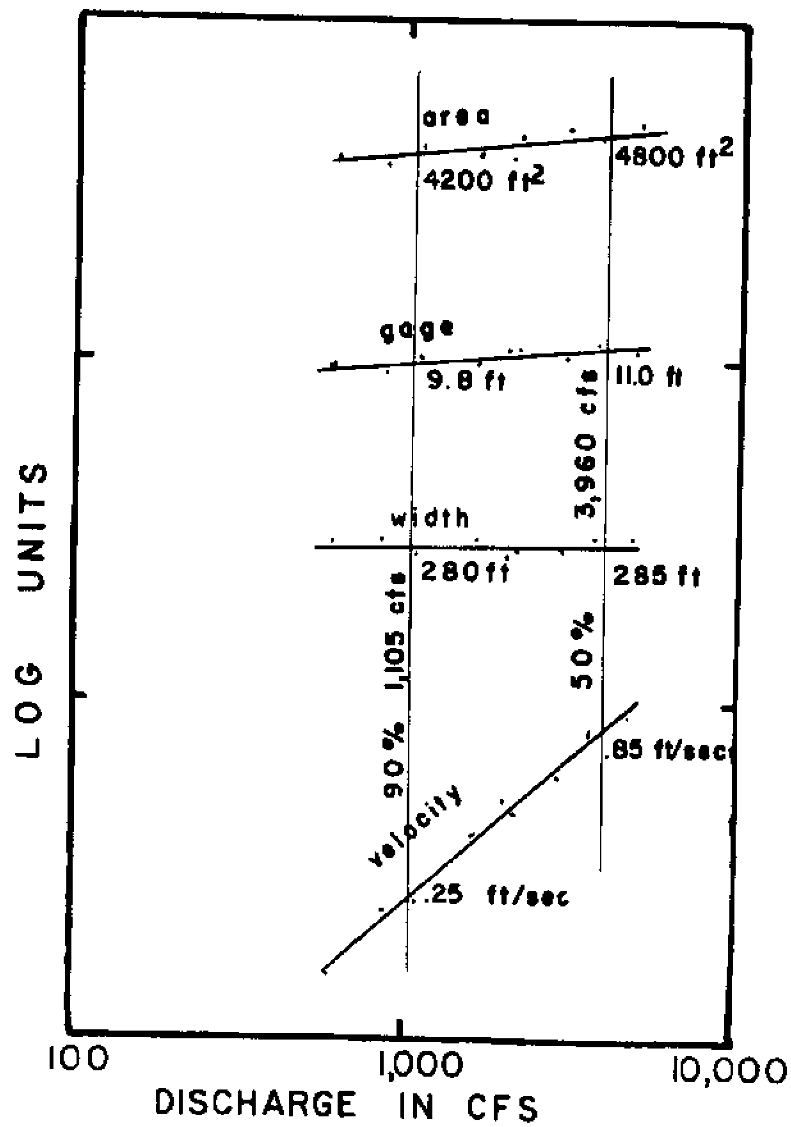


Figure 6.7. Velocity, width, area, and gage height versus discharge for the Green River at Calhoun, KY.

from a dam. If it was located in a section of stream much narrower or wider than the average determined from visual inspection, the cross-sectional area was adjusted by a factor determined from the ratio of the rate of increase of stream width as the dam was approached. To avoid bias to the greatest degree possible, stream width was measured at three points one mile apart immediately above and below the control structures in the reach. The average of these points was used to determine the velocity adjustment factor.

As an example of this technique, the Allegheny River in Pennsylvania has three gaging stations. The highest one, Parkers Landing, is above the head of navigation, and used only for discharge determination. The next one is at Kittanning, mile 45 from Pittsburgh, immediately below dam number 7. The last one is near Natrona on the New Kensington Bridge, mile 19, approximately halfway between dams 3 and 4. The Natrona station was used without modification, but the Kittanning station was in a narrow section of the stream which was unlike the rest of the reach. The distance between dams was 9.4 miles. The average of three widths at the lower end was 1200 feet, while at the upper end the average was 916 feet, or 30 percent larger at the lower end. The velocity used for this reach was, therefore, adjusted downward by half that amount, or by 15 percent.

Information on dispersion coefficients in natural streams was obtained from a number of sources. The best source was actual measurements made through dye dispersion studies. Dispersion coefficients were calculated from dye concentration versus time curves by the change of moment method as given by Fischer (1966)

$$D = 1/2\ u^2 \frac{\sigma^2_{t2} - \sigma^2_{t1}}{\overline{t}_2 - \overline{t}_1} , \qquad (6.8)$$

where t is the mean time of passage from the injection to the sampling point, and σ^2_t is the variance of the concentration versus time curve given by

$$\sigma^2_t = \frac{\int_0^\infty t^2 c\ dt}{\int_0^\infty c\ dt} . \qquad (6.9)$$

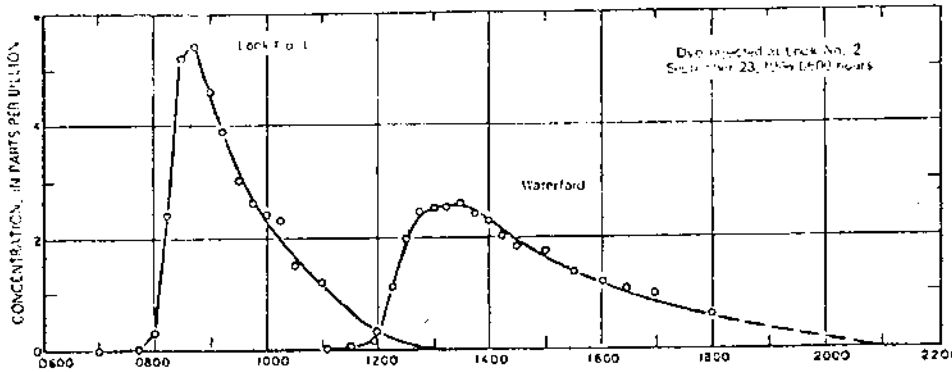
Because of the relatively limited number of dispersion measurements available, calculations were performed by hand. An example of the procedure used is shown in Figure 6.8. Data for this example are from time of travel studies by Shindel (1969a) on the Mohawk River. These dye studies were conducted at high, low and medium discharges at a number of points along the waterway.

In the absence of dispersion measurements, the coefficient prediction release of Fukuoka and Sayre (1973) (equation 3.26) was employed. Information on channel meanders, average bend radius, r_c, and average bend length, L, was obtained from the U.S. Army Corps of Engineers navigation charts.

The friction velocity, $u^* = (\tau/\rho)^{1/2}$, may be evaluated in several ways. The classical definition for open-channel flow is

$$u^* = (\ g\ h\ S_e)^{1/2} , \qquad (6.10)$$

Hudson River-- Dye released at Lock 2 on September 28, 1966



to Lock 1

time	t_o	C
0700	0	0
0800	60	.3
0900	120	5.2
1000	180	2.3
1100	240	1.1
1200	300	0.4

to Waterford

time	t_o	C
1100	240	0
1200	300	.2
1300	360	2.5
1400	420	2.3
1500	480	1.5
1600	540	1.1
1700	600	.9
1800	660	.6
1900	720	.4

$\bar{t}_1 = 154.8$

$\sigma^2_{t1} = 2889.5$

$\bar{t}_2 = 469.9$

$\sigma^2_{t2} = 11{,}493.7$

u=0.75 ft/sec

$$D = 1/2\,(.75)^2 \left(\frac{11{,}493.7 - 2{,}889.5}{469.9 - 154.8} \right) \times 60 \text{ sec/min}$$

$D = 465.7 \text{ ft}^2/\text{sec}$

Figure 6.8. Dispersion coefficient determination from dye dispersion data.

where S_e is the slope of the energy grade line. In uniform flow, S_e is equivalent to the bottom and water surface slope. Where data points are relatively far removed, as with USGS gaging stations, the average slope closely approximates the energy grade line.

When slope information was lacking, u* was evaluated by a method given by Chow (1959). Assuming a logarithmic velocity profile, Chow notes that

$$u^*/u = 3.81\ n\ /\ h^{1/6}\ , \tag{6.11}$$

where n is Manning's roughness coefficient for open-channel flow. Values for n have been obtained experimentally for many channel types and are presented in such references as Chow (1959) and Olson et al. (1966).

Results of the Stage II analysis, metric tons of material required to produce a severe environmental impact (with the assumptions of instantaneous release, complete solubility, and no loss from evaporation or decay) for substances toxic both at 1,000 and 10 ppm, and dilution capacity ratings based on these spill volumes as defined in Chapter V, are presented in Table 6.2. While the spill sizes and dilution capacity ratings are representative values along the river and estuarine systems examined, the same cannot be said for the intracoastal waterway values. Due to the widely varying properties of the intracoastal waterways, dilution capacity varies radically over very short distances. Stage II dilution capacity computations were generally made only at the entrances to land-cut

TABLE 6.2. Stage II Waterway Dilution Capacity Ratings.

Location	Mile	Tons of Material With LC_{50} of 10 ppm	Tons of Material With LC_{50} of 1,000 ppm	Rating
Alabama River	278	48.9	4,890	2.3
	206	39.9	3,990	2.4
	67	64.5	6,450	2.2
Allegheny River	46	175	17,500	1.8
	19	190	19,000	1.7
Apalachicola River	103	60.2	6,017	2.2
Arkansas River	488*	10.9	1,090	2.9
	395	262	26,200	1.6
* Mile 51,	334	239	23,900	1.6
Verdigris River	300	75.3	7,530	2.1
	203	195	19,500	1.7
	118	275	27,500	1.6
Atlantic Intracoastal Waterway	1	32.1	3,210	2.5
	6	33.0	3,300	2.5
	29	5.4	540	3.3
	25 (Rt. 2)	3.5	350	3.4
	102	2.9	290	3.5
	128	2.9	290	3.5
	205	12.7	1,270	2.9
	299	490.0	49,050	1.3
	309	1,300.0	130,000	.9
	310	11.0	1,100	2.9
	321	11.5	1,150	2.9
	330	5.7	570	3.2
	336	1.3	130	3.9
	339	1.5	150	3.8
	342	40.1	4.010	2.4
	375	16.2	1,620	2.8
	397	102.0	10,300	2.0
	401	153.0	15,300	1.8
	435	15.3	1,530	2.8
	459	17.8	1,780	2.7

TABLE 6.2, continued

Location	Mile	Tons of Material With LC_{50} of 10 ppm	Tons of Material With LC_{50} of 1,000 ppm	Rating
Atlantic Intracoastal Waterway (cont.)	472	21.0	2,100	2.7
	480	50.3	5,300	2.3
	505	244.0	24,400	1.6
	543	579.0	57,900	1.2
	557	84.0	8,400	2.1
	573	79.6	7,960	2.1
	576	32.5	3,250	2.5
	586	95.0	9,500	2.0
	592	20.4	2,040	2.7
	597	172.6	17,260	1.9
	606	133.0	13,300	1.9
	615	363.0	36,300	1.4
	621	364.0	36,400	1.4
	626	41.5	4,150	2.4
	659	265.0	26,500	1.6
	670	94.0	9,400	2.0
	685	73.0	7,300	2.1
	694	506.0	50,600	1.3
	700	159.0	15,900	1.8
	715	207.0	20.700	1.7
	721	24.8	2,480	2.6
	729	85.0	8,500	2.1
	740	26.3	2,630	2.6
	776	59.2	5,920	2.2
	793	25.5	2,550	2.6
	822	28.7	2,870	2.5
	840	89.0	8,900	2.1
	987	8.7	875	3.1
	1005	10.7	1,070	3.0
	1013	3.7	376	3.4
	1034	95.0	9,500	2.0
	1035	8.7	870	3.0
	1048	3.6	363	3.4
	1055	5.7	570	3.2
	1080	7.2	720	3.1
Chattahooche River	160	30.4	3,043	2.5

TABLE 6.2, continued.

Location	Mile	Tons of Material With LC_{50} of 10 ppm	Tons of Material With LC_{50} of 1,000 ppm	Rating
Black Warrior-Tombigbee Rivers	339	31.5	3,149	2.5
	213	32.2	3,224	2.5
	117	39.8	3,980	2.4
Columbia River	331	1,157.0	115,750	.9
	292	755.0	75,500	1.1
	189	1,403.0	140,300	.8
	106	1,813.0	181,300	.7
Connecticut River Estuary	45	42.7	4,270	2.3
	19	62.0	6,200	2.2
	0	231.0	23,100	1.6
Cumberland River	381	31.5	3,150	2.5
	308	56.8	5,680	2.2
	212	111.0	11,140	1.9
	149	143.0	14,300	1.8
	89	128.0	12,830	1.9
	30	171.0	17,100	1.7
Delaware River Estuary	126	130.0	13,000	1.9
	119	225.0	22,500	1.6
	95	493.0	49,300	1.3
	90	843.0	84,300	1.1
	78	1,120.0	112,000	.9
	70	1,940.0	194,000	.7
	59	2,260.0	226,000	.6
	40	4,480.0	448,000	.3
Flint River	29	28.0	2,805	2.6
Green and Barren Rivers	149	13.8	1,385	2.9
	100	14.7	1,470	2.8
	63	18.9	1,890	2.7

TABLE 6.2, continued.

Location	Mile	Tons of Material With LC_{50} of 10 ppm	Tons of Material With LC_{50} of 1,000 ppm	Rating
Gulf Intracoastal Waterway West Florida Section (miles from San Carlos Bay Fl)	50	9.4	940	3.0
	60	9.4	940	3.0
Gulf Intracoastal Waterway, Florida panhandle section (miles from New Orleans, LA)	254	3.1	311	3.5
	273	3.1	311	3.5
	313	6.6	660	3.2
	341	20.3	2,030	2.7
	350	99.7	9,970	2.0
Alabama Section	150	2.3	233	3.6
	160	2.3	233	3.6
Gulf Intracoastal Waterway, Texas section (miles from New Orleans)	265	11.0	1,100	2.9
	349	11.0	1,100	2.9
	363	8.8	880	3.1
	376	4.7	470	3.3
	382	2.1	210	3.7
	395	1.8	180	3.7
	401	1.6	160	3.8
	405	1.8	180	3.7
	441	8.2	820	3.1
	450	2.4	240	3.6
Houston Ship Channel (miles from Galveston)	52	4.5	450	3.3
	38	25.7	2,570	2.6
	28	34.9	3,490	2.4
Hudson River below Troy, NY	140	44.0	4,400	2.3
	90	809.0	80,900	1.1
	50	1,086.0	108,600	.9

TABLE 6.2, continued.

Location	Mile	Tons of Material With LC_{50} of 10 ppm	Tons of Material With LC_{50} of 1,000 ppm	Rating
Hudson River continued	0	1,580.0	158,000	.8
Illinois Waterway	291	7.1	715	3.1
	246	119.0	11,940	1.9
	145	138.0	13,850	1.8
	71	162.0	16,190	1.8
James River Estuary	87	53.5	5,350	2.3
	68	131.0	13,100	1.9
	44	454.0	45,400	1.3
	30	795.0	79,500	1.1
	17	1,528.0	152,800	.8
	0	1,222.0	122,200	.9
Kanawha River	54	196.0	19,580	1.7
Kennebec River Estuary	30	21.5	2,150	2.7
	22	102.0	10,200	2.0
	0	700.0	70,000	1.1
Kentucky River	249	15.6	1,562	2.8
	177	13.3	1,337	2.9
	140	13.4	1,340	2.9
	96	15.9	1,588	2.8
	66	10.4	1,045	3.0
	31	16.4	1,637	2.8
Mississippi River (Upper, Miles from Cairo, IL)	865	75.4	7,540	2.1
	726	301.0	30,100	1.5
	633	541.0	54,100	1.3
	512	483.0	48,300	1.3
	364	519.0	51,900	1.3
	203	707.0	70,700	1.2
	178	682.0	68,170	1.2
	110	661.0	66,150	1.2
	44	887.0	88,700	1.1

TABLE 6.2, continued

Location	Mile	Tons of Material With LC_{50} of 10 ppm	Tons of Material With LC_{50} of 1,000 ppm	Rating
Mississippi River, Lower (miles from mouth)	731	794.0	79,400	1.1
	663	820.0	82,000	1.1
	554	1,013.0	101,300	.9
	430	817.0	81,700	1.1
	230	1,267.0	126,760	.9
Missouri River	723	219.0	21,950	1.7
	616	120.0	11,980	1.9
	563	563.0	56,360	1.2
	498	180.0	18,035	1.7
	448	442.0	44,170	1.3
	366	243.0	24,350	1.6
	293	237.0	23,700	1.6
	197	242.0	24,250	1.6
	98	363.0	36,300	1.4
Monongahela River	124	48.0	4,820	2.3
	85	85.0	8,560	2.1
	42	87.0	8,710	2.1
	11	112.0	11,200	1.9
New York State Barge Canal System				
Erie Canal, Lock 30		1.5	154	3.8
Oswego Canal, lock 7		2.2	226	3.6
Seneca R. at Baldwinsville		6.1	612	3.2
Mohawk R. at Little Falls		3.9	391	3.4
Mohawk R. at Cohoes		24.0	2,400	2.6
Hudson R. at Green Island		91.4	9,140	2.0
Ohio River miles from Pittsburgh.	12	483.0	48,300	1.3
	155	385.0	38,500	1.2
	184	659.0	65,900	1.2
	311	654.0	65,400	1.2
	322	675.0	67,500	1.2
	408	788.0	78,800	1.1

TABLE 6.2, continued

Location	Mile	Tons of Material With LC_{50} of 10 ppm	Tons of Material With LC_{50} of 1,000 ppm	Rating
Ohio River continued	470	578.0	57,800	1.2
	607	545.0	54,500	1.2
	629	855.0	85,500	1.1
	903	965.0	96,500	1.0
	944	1,194.0	194,000	.7
Okeechobee Waterway miles from St. Lucie inlet, FL.	15	2.2	226	3.6
	77	2.2	226	3.6
Penobscot River Estuary	14	63.0	6,300	2.2
	0	322.0	32,200	1.5
Sacramento River miles from Sacramento CA	89	17.3	1,730	2.8
	63	13.6	1,360	2.8
	34	14.1	1,410	2.8
	20	37.3	3,730	2.4
	0	56.8	5,680	2.2
St. Johns River, Florida	123	24.4	2,440	2.6
	57	95.0	9,500	2.0
	30	385.0	38,500	1.4
San Joaquin River CA	120	4.4	441	3.3
	40	9.6	957	3.0
Savannah River	129	27.0	2,736	2.6
	65	37.0	3,786	2.4
	3	275.0	27,500	1.6
Snake River	140	100.0	9,990	2.0
	10	241.0	24,120	1.6

TABLE 6.2, continued.

Location	Mile	Tons of Material With LC_{50} of 10 ppm	Tons of Material With LC_{50} of 1,000 ppm	Rating
Tennessee River	651	121.0	12,090	1.9
	430	146.0	14,600	1.8
	334	286.0	28,600	1.5
	257	258.0	25,800	1.6
	190	127.0	12,700	1.9
	0	247.0	24,700	1.6
Willamette River	119	59.0	5,910	2.2
	84	70.0	7,005	2.1
	37	132.0	13,223	1.9

sections in order to estimate this parameter over areas where available dilution water is the lowest. The intracoastal waterway values are included for completeness, but should be taken only as point values from which wide departures exist. Information on specific waterways, dispersion coefficients and adjustment rationale are presented in Appendix A.

CHAPTER VII

SUMMARY AND CONCLUSIONS

The purpose of this study is to develop practical means by which the spill dilution capacity of a waterway might be employed as a management tool to reduce risks associated with the carriage of oil and hazardous materials. It was desired that the procedure be applicable to both tidal and non-tidal portions of waterways, and that the data necessary be available without extensive field collection.

Two levels of spill dilution capacity were developed in order to provide users with a maximum degree of flexibility in application. The first level involves the determination of the most probable flow past a spill site available for dilution. The second level uses this information combined with waterway hydraulic parameters in the one-dimensional dispersion model first developed by Taylor (1953, 1954) to estimate the amount of spill required to produce a given concentration in the analysis unit. This spill volume is then a relative measure of dilution capacity which considers probable dispersion in the waterway, and it can also be used as a design tool to estimate safety factor requirements for a given material and area.

The background of the problem, including a review of current regulatory efforts, is presented in Chapter I. A qualitative description of the range of different types of hazardous materials, including cryogenic gases and insoluble materials, is presented in Chapter II. An intensive review of the literature on modeling the dispersion of solutes, the basis of this analysis, is presented in Chapter III.

In Chapter IV, the water quality aspects of hazardous materials spills are reviewed in depth. An analysis of the dynamics of toxic action is conducted with the aim of determining a suitable criterion for spill impact. It was found that maximum concentration in the analysis unit was such a suitable criterion.

Chapter V presents the development of dilution capacity analysis procedures. The reference pollutant used is a conservative solute. The procedures developed meet the criteria of (1) defining the most important elements of spill dilution capacity, (2) making use of readily available information, and (3) presenting a readily usable approach to water pollution risk reduction. In Chapter VI, these procedures are applied to the major inland and Intracoastal Waterways and the results are presented in tabular form.

Waterway dilution capacity information presented in Chapter VI could be used in a variety of ways to reduce water pollution risk. A few of these will be presented here to provide a framework for evaluation.

The relative dilution capacity ratings presented in Table 6.1 could be used directly as the basis for shipping regulations. This could either take the form of a relaxing of requirements on Class 0 waterways where the pollution risk is lowest, or the application of more stringent requirements where the risk is greatest, for example Class 2 or higher.

Stage I is a relative rating scale, patterned after the NAS Hazardous Material Rating System. A logical method of using Stage I is in combination with the NAS aquatic toxicity rating. The sum of the NAS aquatic toxicity rating for a particular commodity and the Stage I rating for a waterway would provide a better measure of the actual water pollution risk to the waterway than would either hazard rating alone. A description of how this might actually apply in practice is given as Appendix B.

The second level of dilution capacity analysis (Stage II) also provides a relative measure of waterway vulnerability to spills, except that Stage II takes into account the dispersion characteristics of non-tidal waterways, and presents the information in terms of the amount of spill required to produce either a 10 or 1000 ppm concentration in the analysis unit. The Stage II tonnages presented in Table 6.2 are then used to define a dilution capacity ranking system similar to the Stage I system. The use of the Stage II ratings is identical with Stage I, except that in non-tidal systems the ratings reflect the dispersive characteristics of the waterway.

Another way that Stage II presentation of dilution volume might be used is as an indicator of critical spill volumes for a waterway.

This would be in conjunction with a decision on the amount of environmental impact that is critical for an area. For example, if it is determined that twenty-five miles is a reasonable upper bound on the length of waterway that could be severely damaged and still recover without unacceptable environmental damage or public outcry, shipping regulations could be designed to prevent a release larger than the Table 6.2 value, adjusted for the toxicity of the material. Another description of possible application of Stage II is given in Appendix C.

Stage II information also may have applications in land use planning. The location of a large petrochemical complex, with attendant possibilities for serious spills, should take into account the probable spill dilution capacity of nearby waterways.

Subjects for Future Research

In the process of determining selective application of safety precautions to minimize water pollution risk, the subject of waterway "value" may arise. The underlying assumption in this analysis has been that all natural systems are of equal value and hence should be protected according to the degree of risk to which they are exposed.

It is conceivable that this situation would be altered, however, because some waterways obviously have more value to the public than others. For example, if the public were asked whether the

Houston Ship Channel and Lower Galveston Bay should be afforded equal expenditures for environmental protection, many would probably answer in the negative. The Houston Ship Channel has almost no recreational, aesthetic, or fishery value, while Lower Galveston Bay is heavily utilized for all these functions.

The value systems that could be applied to waterways might be based on economic, ecological, aesthetic, or political determinants. Techniques for economic valuation of waterways as fishery and recreational resources have been developed for a number of water resource projects. A valuation could be placed on a waterway based on the role of the waterway in the ecology of a larger system. Without a high level of knowledge of ecological interactions, however, such a valuation would be highly speculative. Techniques for quantifying aesthetic value of a waterway, based on the personal preferences of the community, have been developed. Application of these techniques by Dearinger, et al. (1973) in a study of streams in Kentucky, proves the complexity of the approach, however.

To a certain extent, political processes encompass all of the previously mentioned determinants of value, in that economic, ecological, and aesthetic factors all determine political opinion. Additional inputs are also considered in political opinion, however, such as the effect of shipping costs on the economy of an area.

The benefits and costs associated with decisions involving environmental and political factors must obviously be weighed carefully. This area of combined environmental and political impact needs much careful research in order to define priorities and alternatives.

Whether or not waterway value is employed as a measure of the amount of safety required, the spill dilution capacity of the waterway, quantified in this research, will be an important factor in determining risk to the waterway.

Another area needing further research is the seasonal variability of waterway flow. This analysis has recognized the temporal variability in flow, and has used the technique of flow duration to determine a most probable discharge for a stream. The actual discharge of the stream at a given time will vary from this value by a considerable margin, depending on a number of geographical factors including the amount of flow regulation upstream. Because of this variability, rivers whose low flows bring them into a different dilution volume rating class, as indicated by the flow which is exceeded 90 percent of the time, are marked for special consideration. Future work should consider seasonal variability; however, the application of such a system should be weighed against obvious difficulties in use and enforcement.

Some hazardous materials which are not notably toxic may still pose a severe water pollution threat by exerting a high biochemical oxygen demand (BOD) on the waterway. The vulnerability of the waterway is a function of residence time in the system and amount of natural reaeration as well as dilution capacity. Residence time is important because the decay of organic materials generally involves an initial lag time while the microbial population in the waterway increases to a point where rapid oxidation can proceed.

The rate of natural reaeration in the waterway also is important because it determines the critical rate of oxygen depletion allowed for the stream. If the rate of oxygen depletion is greater than the rate of reaeration, oxygen depletion, and the associated decimation of the aquatic community will result. An analysis of waterway vulnerability to spills of high BOD substances is a logical next step in waterway vulnerability analysis. Much of the information collected in this study would be directly applicable to such a project.

Conclusions

A number of conclusions may be drawn from this investigation. It has been determined that spill dilution capacity strongly affects the impact of a spill of toxic material. Based on most probable concentration, spill dilution capacity approaches the importance of material toxicity in determining the severity of spill impact.

It has been concluded that adequate data exist to characterize the spill dilution capacity of major American waterways in terms of hydraulic parameters.

An analysis of toxic action on finfish of several pollutants revealed that for distances sufficiently far removed from the spill site, maximum concentration in the water was a suitable parameter upon which to base an analysis of stream dilution capacity.

A system was developed whereby spill dilution capacity was quantified in tabular form, amenable to direct utilization by regulatory agencies in a national program of OHM water pollution risk reduction.

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APPENDIX A

Appendix A includes descriptions of the waterways evaluated in this study, tabular summaries of the flow data used, and explanations of how the dilution capacity ratings were assigned. Waterways are arranged alphabetically, with the exception that smaller tributaries are listed as part of major river systems. For example, the Verdigris River, part of the Arkansas River system, is listed in the Arkansas River summary.

Where blanks exist in the data summary tables, no data were available. Data in parenthesis were of questionable quality, obtained from limited records or by extrapolation of curves.

Alabama River

The Alabama River is navigable from its confluence with the Tombigbee River to Montgomery, AL, a distance of 278 miles. Plans call for the extension of the waterway to include the Coosa and Tallapoosa Rivers.

Project depth for the waterway is 9 feet and project width is 200 feet.

Stage I

All the stations on the Alabama River indicate a Class I dilution capacity rating.

Stage II

No dispersion coefficient measurements were available for the Alabama River. The method of Fukuoka and Sayre was employed, with r_c and L equal to 2,304 and 6,546, respectively. With n = .03, the following spill volumes required to produce 1,000 ppm, 25 miles downstream, were computed:

Mile	D	w/h	Tons
278	644.2	41.9	4,892
206	548.3	27.7	3,988
67	988.4	57.8	6,648

Alabama River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Montgomery, Ala. 02420000	90%	7,600	5,200	1.4	500	99.9
River Mile 278.4	50%	14,500	7,200	1.9	550	103.9
Selma, Ala. 02423000	90%	8,700	5,400	1.7	380	65.0
River Mile 205.5	50%	16,000	7,000	2.3	440	69.8
Clairborne, Ala. 02429500	90%	9,650	5,000	1.8	600	10.4
River Mile 66.8	50%	18,725	8,500	2.2	700	15.9

Allegheny River

Navigation begins at East Brady, PA, and extends for 72 miles southwestward to its confluence with the Monongahela River at Pittsburgh. Project depth is 9 feet and width is 200 feet. There are 9 locks (including the Emsworth dams on the Ohio), each measuring 56 feet wide and 360 feet long.

<u>Stage I</u>

Although the 50% duration flow at the lowest station is in excess of the class 2 limits, the greater part of the river is in the class 2 range. Accordingly, a class 2 rating (10,000 - 1,000 cfs) was assigned.

<u>Stage II</u>

No dye dispersion measurements were available. Because the river has a small slope during low flow periods, the dispersion coefficient prediction method of Fukuoka and Sayre (1973) was deemed most suitable. From navigation charts, mean values for bend radius and bend length of 5585 and 12,140 feet respectively were determined. With n=0.03, and the information at the gaging stations, the following amounts of spill required to produce 1000 ppm concentration 25 miles downriver were computed.

Mile	D	W/h	Tons
45.8	5,525	96.0	17,500
19.0	1,090	49.7	19,000

Allegheny River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Parker Landing, Pa.* 03031500	90%	1,450		1.1		846.94
River Mile 83.4	50%	7,490	3,200	2.3	790	848.44
Kittanning, Pa. 03036500	90%	1,830	5,000	.3	700	
River Mile 45.8	50%	8,860	7,000	1.2	820	784.17
Natrona, Pa. 03049500	90%	2,380	13,000	1.6	820	
River Mile 19.0	50%	11,600	14,200	.82	840	747.71
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					

*Above head of navigation

Apalachicola, Chattahoochee and Flint River System

The Flint River is navigable from Bainbridge, Georgia to Lake Seminole, a distance of 29 miles. There is a gaging station at Bainbridge, Georgia.

The head of navigation on the Chattahoochee River is Columbus, Georgia, 155 miles above the Woodruff lock and dam. It flows southerly to the Lake Seminole, joining the lake approximately at mile 15 above the Woodruff Dam. There is a gaging station at Columbus, Georgia, one half mile below Eagle and Phenix Dam at mile 159.9. The Chattahoochee flows into an impoundment formed by the Walter F. George lock and dam at mile 75. It then flows on to Lake Seminole at approximately mile 15.

The Apalachicola River is formed by the drainage from Lake Seminole at Jim Woodruff Lock and Dam and flows 104 miles to Apalachicola Bay at the City of Apalachicola, Florida. A gaging station is located 0.6 miles below Woodruff Dam on U.S. Highway 90 Bridge.

Stage I

The Chattahoochee and Flint Rivers fall into class 2 as indicated by their 50% flows. The Apalachicola River from Lake Seminole to Apalachicola Bay is on the other hand, in class 1.

Stage II

No dispersion coefficient measurements are reported for these rivers. Using the method of Fukuoka and Sayre, with n = .03, gives the following values for the spill size required to produce 1000 ppm.

	D	W/h	Tons
Apalachicola River at Chattahoochee	385.8	52.1	6,017
Chattahoochee River at Columbus, Georgia	2,176.3	53.7	3,043
Flint River at Bainbridge	100.3	22.8	2,805

Apalachicola, Chattahoochee and Flint River System

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Chattahooche R. at Columbus, Ga.	90%	1,750	1,800	0.95	(360)	190.14
02341500	50%	5,160	2,500	2.0	(365)	191.34
River Mile 159.9						
Flint R. at Bainbridge, Ga.	90%	3,330	6,000	.55	400	
02356000	50%	6,340	7,000	.85	400	
River Mile 29.0						
Apalachicola R. near Chattahooche, Fl.	90%	9,644	7,600	1.25	700	43.2
02358000	50%	16,825	10,500	1.6	740	46.8

Arkansas River Navigation System

The Arkansas River Waterway consists of the Arkansas Post Canal (White River) for 10 miles from the Mississippi River, Arkansas River from its mouth to Muskogee, Oklahoma, and the Verdigris River from its mouth at Muskogee to Catoosa, Oklahoma. Navigable length of the Arkansas River is 397 miles while the Verdigris River is navigable for 51 miles. The Arkansas River is also scheduled to be opened for navigation to Tulsa, Oklahoma in the near future.

Project depth for the system is 9 feet. Project width is 300 feet on the Arkansas Post Canal, 250 feet on the Arkansas River and 150 feet on the Verdigris River. A system of 17 locks and dams control flow and provide power generating capacity for the area. Strong daily and weekly fluctuations occur in river flows due to the effects of water use for power generation.

Calculations

Stage I

The Arkansas River from its mouth to Muskogee (Mile 395) falls into dilution class 1 (10,000 - 100,000 cfs). The Verdigris River was assigned class 2, although the upper end at Catoosa is indicated to be in class 3.

Stage II

Dispersion coefficient information was available from a USGS time of travel summary sheet giving duration of dye cloud at a downstream station. On the Verdigris River, mainly a canal cut through the valley, with most of the meanders removed, a D/hu* value of 50 was used. This value is representative of the values reported by Fischer (1973) for this type of waterway.

The prediction method of Fukuoka and Sayre was used on the upper end of the Arkansas River above the reach where measurements were available. In this area, r_c and L were, respectively, 4,700 and 10,000 feet. With the indicated data sources, the following spill amounts required to produce 1,000 ppm, 25 miles downstream, were computed.

Mile	D	w/h	Tons
488[a]	11.4[b]	32.5	1,090
395	951.5[c]	67.0	26,200
334	1153.0[d]	26.2	23,940
300	1153.0[d]	37.0	7,530
203	3480.0[d]	73.6	19,550
118	1890.0[d]	62.4	27,530

[a] Verdigris River at Catoosa, Oklahoma

[b] D/hu* = 50

[c] Fukuoka and Sayre

[d] Dispersion coefficients from USGS data printout

Arkansas River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Catoosa, Okla.	90%	250				
River Mile 488.3*	50%	800	(3900)	(.2)	356	532
Muskogee, Okla.	90%	2,200				
River Mile 394.8	50%	11,000	(18000)	(.61)	1100	487
Sallisaw, Okla.	90%	4,600				
River Mile 334.0	50%	11,500	(16100)	(.71)	650	412
VanBuren, Okla. 07250550	90%	4,900	6,900	.7	530	
River Mile 300.4	50%	12,500	7,600	1.6	530	391
Lake Dardanelle, Arkansas	90%	6,000				
07258000 River Mile 203.5	50%	16,000	(11000)	(1.5)	900	284
Little Rock, Ark. 07263450	90%	6,500				
River Mile 118.5	50%	20,500	18,000	1.1	1060	231

* Mile 51 Verdigris River

Atlantic Intracoastal Waterway--Virginia

The AIW begins in Norfolk, VA, and runs southward along the Elizabeth River. At mile 7, the Dismal Swamp route (route 2) branches off, and route 1 continues through a dredged upper Elizabeth River and through a land-cut section to mile 30. From mile 30 to 34, the North Carolina border, the AIW proceeds through the upper reaches of the North Landing River.

Stage I

Tidal range at Norfolk is 2.8 feet, but drops off sharply along the Elizabeth River. At mile 29, there is no tide indicated. Using a tide range of 1 foot, a mean surface width of 300 feet, yields discharge in the class 3 range. Tidal current data for Norfolk yield flows in the class 2 range at the northern extremity of the AIW.

Mile	Area	Vel	Q
1	21,000	.3 .3	5,040
6	10,000	.6 .7	5,200

At mile 30, cubature yields 850 cfs with a 1 foot range assumed. With the smaller tidal range indicated, the flow would be considerably less. Except for the northern portion, discharge in the AIW is in the class 3 range.

The Route 2 section from mile 7 to the N. Carolina border at mile 25 has even less flow due to its reduced size. Using a 1 foot tide, the 200 foot wide waterway would have a discharge of 560 cfs at the ends or 280 cfs average tidal flow.

Because of these values, a class 3 rating was assigned the AIW in Virginia.

Stage II

Using the flow values and the areas from tidal cubature, the following spill sizes required to produce 1,000 ppm in the tidal excursion were calculated.

Mile	Tidal Excursion	Tons
1	1.0	3,210
6	2.2	3,300
29	1.2	540
25 (route 2)	1.3	350

Atlantic Intracoastal Waterway--North Carolina

The AIW in North Carolina runs from mile 34 to 341. The great majority of this distance is through open water; however, a portion is cut through marsh land and small protected sounds.

Stage I

Mile 34 - 102: Class 0. This section proceeds through North Landing River and Currituck Sound, both large waterways. After a short land cut section from mile 48-52, the AIW proceeds into the North River and Albemarle Sound to the Alligator River. With the exception of miles 48-52, this section of the waterway is through large (greater than 1 mile wide) waterways.

Mile 102 - 128: Class 3. The Alligator River becomes shallow and narrow at mile 102, and follows a land-cut section where it joins the Pungo River at mile 128. Tidal ranges in this section are less than one half foot. Using 300 ft. as the top width of the waterway and a .5 foot tidal range uniform along the reach yields discharges at the mouths of 460 cfs. The average discharge along the reach is then 230 cfs.

Mile 128 - 185: Class 0. The Pungo River is followed until the AIW joins the Pamlico River at mile 145. At mile 150, the AIW enters Goose Creek for a 10 mile stretch of land cut section before joining open water in the Bay and on into the Neuse River. With the exception of mile 150-160, this stretch of AIW is all in large rivers and open bay areas, indicating large spill dilution capacity.

Mile 185 - 205: Class 2. This portion is a narrow land-cut section terminating at Beaufort, NC. Tidal range at Beaufort is 2.5 feet. Using the average surface width of 300 feet and mid-length of 10 miles, the tidal discharge computed at Beaufort is 2,000 cfs. The average value is then 1,000 cfs or in class 2.

Mile 205 - 297: Class 2. This section is composed of Bogue Sound to Bogue Inlet at mile 230, where it narrows to a series of marshes with the AIW dredged through. Inlets allow tidal flushing at miles 237, 245, 270, 275, 280, 285, and 294. Prediction of tidal discharges is difficult because of the complex marsh systems and variable surface areas. A class 2 dilution capacity rating was assigned based on the approximate size, tidal range and proximity to the ocean.

Mile 297 - 309: Class 1. This section is relatively open water of the Cape Fear River. Tidal Current data along the river indicate a class 1 or lower rating.

Mile	Area	Vel	Q
299	52,000	1.7 2.0	77,000
309	72,000	3.1 4.0	204,000

Mile 309 - 341: Class 2. This portion of the AIW is land-cut sections with inlets located at miles 321, 330, 336, and 339. Tidal range is 2.5 feet along the coast. Using this tidal range as uniform along the reaches, and a 300-foot surface width, the following values were obtained.

Mile	Discharge
309	1,750
321	1,800
330	900
336	210
339	240

Although tidal-induced discharges are small and indicate a class 3 rating, the dilution capacity rating was adjusted to class 2 because of the proximity of all sections to the open ocean.

Stage II

Using the flow values calculated in Stage I, the following spill sizes required to produce 1,000 ppm in the tidal excursion were calculated.

Mile	Spill Tonnage
102	290
128	290
205	1,270
299	49,050
309	1,100
321	1,150
330	570
336	130
339	150

Atlantic Intracoastal Waterway--South Carolina

The AIW follows the South Carolina coast from mile 341 (from Norfolk) to mile 576. The northern portion of the AIW to Charleston is primarily land-cut section, while the southern end is characterized by the relatively open water in the various sounds and inlets, connected by short stretches of land-cut waterway.

Stage I

A combination of tidal current and tidal cubature data was employed to obtain the following flows for the South Carolina AIW.

Mile 341 - 464: Class 2. The AIW crosses Little River Inlet at mile 342 and proceeds along a land-cut section to mile 375, where it joins the upper reaches of the Waccamaw River. Average freshwater flow in the Waccamaw is at Longs, SC, 60 miles above mile 375, and is 1200 cfs. The Waccamaw gets larger as it approaches Winyah Bay. Leaving Winyah Bay at mile 411, it follows a land-cut section which crosses the north and south Santee Rivers. It then runs adjacent to and through Cape Romain National Wildlife Refuge (primarily marsh land) until it enters the Cooper River at mile 464.

Discharges at the following points were computed:

Mile	Area	Vel		Q	
342*	3,600			6,300	average for reach 2,200
375*	4,000			2,540	
397	26,400	.6	.9	16,000	upper Winyah Bay
401	31,200	.7	1.2	24,000	
435*	5,000			2,400	
459*	4,500			2,800	

*by tidal cubature

Mile 465 - 576: Class 1. Leaving the Cooper River at Charleston (mile 470), the AIW cuts through to the Stono at mile 472 and to the Wadmalaw River at mile 488. It then follows the Wadmalaw to the N. Edisto at mile 495, then into the Dawho River to 504, where it joins the S. Edisto. At mile 511, a short cut connects the AIW to the Ashepoo River, through 5 miles of the Ashepoo-Coosaw cut-off and into the Coosaw River. At mile 529 it leaves the Coosaw and enters the Beaufort River, where it follows into Port Royal Sound at mile 548. Skull Creek, mile 553 to 558, connects the AIW to the Calibogue Sound. Leaving Calibogue Sound at mile 568, it connects through the New and Wright Rivers to the Savannah River at mile 576.

The following tidal discharge values were obtained at points along the reach of the AIW:

Mile	Area	Vel		Q
472	2,500	1.6	1.9	3,300
480	6,000	1.3	2.0	7,900
505	24,000	1.8	2.2	38,400
543	69,000	1.5	1.8	91,000
557	17,400	.7	1.2	13,200
573	11,200	1.2	1.6	12,500

Stage II

Using the discharge information from Stage I, the following spill sizes required to produce 1,000 ppm in the tidal excursion were computed.

Mile	Tidal Excursion (Miles)	Tons
342	7.4	4,010
375	2.7	1,620
397	2.6	10,200
401	3.2	15,300
435	2.0	1,530
459	2.6	1,780
472	5.6	2,100
480	5.6	5,030
505	6.8	24,400
543	5.6	57,900
557	3.2	8,400
573	4.7	7,960

Atlantic Intracoastal Waterway--Georgia

The AIW follows a sinuous path through a series of sounds and short tidal rivers from mile 576 (from Norfolk) to mile 714 at the St. Marys River. Most of the waterway is through relatively large river or wide channel sections, frequently open to the ocean at tidal inlets. Tide range along the Georgia coast is considerable, generally 5 to 7 feet, which produces large tidal flows and rapid flushing of the AIW.

Stage I

Because tidal current information was available at many locations along the Georgia AIW, tidal cubature was not often required to characterize the flow.

Mile 576 - 596: Class 2. The AIW runs from the Savannah River through Elba Island Cut to the Wilmington River, to the Skidaway River at mile 586. It then proceeds through Skidaway narrows at mile 592 into the Vernon River at mile 596. The following values were obtained by cubature and from tidal current information to characterize the flow in this reach:

Mile	Area	Vel	Q
576	7,200	.7	5,100*
586	15,600	1.0 1.4	15,000
592	4,000	.9 1.1	3,200

*computed by tidal cubature

Mile 597 - 714: Class 1. The AIW enters the Vernon River at 597 and thence into the Ogeechee River, through the Florida Passage at 606, through the Bear River (615) and on to St. Catherines Sound at 617. It then enters the North Newport River at mile 621 and into Johnson Creek where the flow in the middle at mile 626 has been estimated. It then enters Sapelo Sound and joins the Front River at mile 639. From there, it flows through Old Teakettle Creek to Doboy Sound at 647. The North and Little Mud Rivers then carry the AIW to Althmah Sound at mile 655, which it follows to the Mackay River at mile 670. It follows the Mackay into St. Simons Sound at mile 682, behind Jekyll Island to St. Andrews Sound and on into the Cumberland River, which is followed until the AIW enters the St. Marys at mile 712.

This reach is characterized by wide tidal inlets and short or nonexistent land-cut sections connecting the sounds. Tidal current information determined along this reach indicates the flows are generally in the class 1 range.

Mile	Area	Vel		Q
597	24,200	1.1	1.7	27,100
606	18,000	1.3	1.6	20,900
615	39,600	1.2	2.0	57,000
621	55,000	1.2	1.4	57,200
626	10,000	.8	.9	6,500
659	36,000	1.0	1.9	41,700
670	15,300	.9	1.5	14,700
685	12,000	1.0	1.4	11,500
694	71,000	1.3	1.5	79,500
700	24,000	1.3	1.3	25,000

Stage II

Using the Stage I data, the following spill sizes required to produce 1,000 ppm in the tidal excursion were computed:

Mile	Tidal Excursion Miles	Tons
576	3.0	3,250
586	4.1	9,500
592	3.4	2,040
597	4.75	17,260
606	4.9	13,300
615	6.1	36,300
621	4.4	36,400
626	2.7	4,150
659	4.9	26,500

(continued)

Mile	Tidal Excursion Miles	Tons
670	4.1	9,400
685	4.1	7,300
694	4.7	50,600
700	4.4	15,900

Atlantic Intracoastal Waterway--Florida Section

The AIW enters Florida at mile 714 (from Norfolk, Virginia). It procedes up the Amelia River and enters a land cut section at mile 720. It then enters the South Amelia River at mile 723, passes through Nassau Sound and enters a narrow section cut through Sawpit Creek. It follows this narrow section from mile 729.5 to 739.5 where it enters the St. Johns River.

From mile 740 to 776 it is all land cut section although it gets wider in the lower section as the AIW merges with the Tolomato River ending up at St. Augustine, Florida. After St. Augustine, the AIW follows the Matanzas River for 8 miles and then enters land cut sections until it gradually enters Tomoka Basin at mile 822. Tomoka Basin is a shallow bay averaging over one half mile wide. The AIW now follows a path through marsh islands and shallow bays behind the barrier island, past New Smyrna Beach at mile 852 and into Mosquito Lagoon. This section of the AIW is through the Merritt Island National Wildlife Refuge. After mile 875, the AIW enters part of the Indian River, which is a bay protected by barrier islands. It follows this bay until it reaches St. Lucie Inlet at mile 987.

From St. Lucie Inlet, the AIW proceeds through land cut sections alternated by narrow bays to Jupiter Inlet at mile 1005. The AIW then proceeds through land cut sections to Lake Worth at mile 1013. At mile 1034, it again leaves open bay sections and enters a land cut stretch until mile 1066 at Port Everglades and re-enters land cut sections until it enters open bay at mile 1080 (Biscayne Bay). The rest of the AIW to Miami is open bay.

Stage I

The dilution capacity of the AIW waterway varies sharply from point to point along its length. Dilution class values based on the following calculations were assigned.

Mile 714-729: Class 1. Data from the tidal current tables indicate that discharge ranges from a high of 32,500 cfs at Fernandia Beach to 3,900 cfs in the interior land cut section (mile 721) and back to 13,400 at mile 729. The average value tidal current data for the reach is 13,500 cfs.

Mile	Area	Vel fld	Vel ebb	Q
715	25,000	1.4	1.6	32,500
721	3,600	1.4	1.4	3,900
729	12,000	1.4	1.4	13,400

Mile 729.5-739.5: Class 2. Using tidal cubature, and an indicated 5 foot mean semidiurnal tide range at each end of the cut, a discharge at the ends of the 10 mile reach of 2,350 cfs was computed. The average value is then 1175.

Mile 740-776: Class 2. A mean tide range of 4.5 feet exists at each end of this 36 mile reach. Using 18 miles as the tidal effects modal point, a discharge of 4,125 cfs at St. Johns (mile 740) and 9,300 cfs at St. Augustine were computed. The average Q over the reach is then 3350 cfs.

Mile 793-822: Class 2. A tidal range of 4.4 feet exists at the northern end of this reach while the range is essentially 0 in the northern end of the Tomoka Basin to the south. Using one half the tide range yields a tidal discharge at the northern end of 4,500 cfs, or an average value over the reach of 2,250 cfs.

Mile 822-840: Class 1. This stretch of the AIW is typically one half mile wide. Predicting flow is quite difficult because of numerous earthen bridge structures which limit flow to a narrow bridge opening and because of the separation from tidal effects due to its length. Because of the relatively low flow along its length, even with its large size, this stretch is assigned a class 1 rating.

Mile 840-858: Class 1. This stretch is a series of moderately wide bays with marsh islands scattered along the route. The tidal range at the northern end (Ponce de Leon Inlet) is only 2.3 feet. The indicated flow at the northern end of the stretch is 14,000 cfs. Due to the large surface area of this stretch, it is kept in class 1 even though average flow would be in class 2.

Mile 858-987: Class 0. This long stretch of the AIW is through open bays with a width ranging from one half to two miles.

Mile 987-1005: Class 3. This 18 mile stretch from St. Lucie Inlet to Jupiter Inlet is primarily land cut with several relatively narrow bays along the way. The tide range at St. Lucie is 1 foot while at Jupiter it is 2 feet. By cubature, a Q of 2,000 cfs at Jupiter and 1060 at St. Lucie is indicated. The average Q along the route is then 790 cfs.

Mile 1005-1013: Class 3. This section is almost entirely land cut. Tidal range at the northern end is 2 feet while Lake Worth on the southern end is 2.1 feet (at the port of Palm Beach). Cubature yields a mean tidal discharge at the ends of 590 cfs, indicating an average value over the reach of 295 cfs.

Mile 1013-1034: Class 1. Although Lake Worth is large, averaging one half mile wide, the flows entering and leaving are quite restricted, below to a value less than would be indicated by cubature (15,000 cfs at each end). A compromise dilution capacity rating of 1 was assigned.

Mile 1034-1080: Class 3. This stretch is almost entirely land cut. Openings to the ocean are at miles 1048, 1055, 1066 and 1088, where the mean tidal range is 2.3 feet. Using tidal cubature on each stretch between openings to the ocean yields tidal discharge values at each end of 1054, 570, 892 and 1135 cfs respectively. Taking the average of these discharge values over the length of each reach yields Q values in the class 3 range.

Mile 1080-1089: Class 1. This section is entirely open bay waters until the AIW reaches Miami.

Stage II

Using the mean tidal discharge values calculated in Stage I, the following spill volumes required to produce 1,000 ppm in the tidal excursion were calculated.

Mile	Tidal Excursion	Tons
715	5.5	20,700
721	4.6	2,480
729	4.7	8,500
739.5	4.0	1,500
740	5.1	2,630
776	4.8	5,920
793	5.4	2,550
822	6.8	2,870
840	5.7	8,900
987	3.5	875
1005	3.2	1,074
1013	4.0	376
1034	4.8	9,500
1035	4.2	870
1048	5.0	363
1055	4.0	570
1080	4.3	720

Black Warrior and Tombigbee River System

The Black Warrior-Tombigbee River System lies entirely within the state of Alabama. Total navigable distance, including the Sipsey, Mulberry and Locust Forks of the Black Warrior River, is 466 miles. Project depth for the system is 9 feet, while width is 200 feet. Navigation pool levels are controlled by six dams.

Stage I

All the stations on the waterway have a median flow in the class 2 range (10,000 - 1,000 cfs).

Stage II

No dispersion measurements were available for the waterway. The dispersion coefficient prediction relation of Fukuoka and Sayre was used, with n taken as .03. Using this relation, with r_c and L 1,926 and 5,566 feet, respectively, the following spill volumes required to produce 1,000 ppm, 25 miles downstream, were computed.

Mile	D	w/h	Tons
339	51.8	17.3	3,149
213	243.8	24.7	3,224
117	190.2	23.6	3,979

Black Warrior and Tombigbee River System

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Black Warrior at Tuscaloosa, Ala.	90%	810	5,900	.12	340	123.4
024650 River Mile 338.9	50%	3,065	7,500	.4	360	124.2
Tombigbee River at Demopolis L&D	90%	1,920	5,400	.36	360	73.5
02467000 River Mile 213.4	50%	8,440	6,500	1.35	400	74.5
Tombigbee River at Jackson L&D	90%	2,840	7,300	.4	405	46.3
02469761 River Mile 116.6	50%	8,800	8,200	1.1	440	46.5

Columbia, Snake and Willamette River System

Navigation on the Columbia River extends from its mouth to 48 miles above McNary Dam, a distance of 340 miles. The Snake River is navigable from its confluence with the Columbia just below Pasco, WA, to above Lewiston, ID, a distance of 140 miles. Head of navigation on the Willamette River is Corvallis, OR, 132 miles from its mouth near Portland, OR.

Stage I

Columbia

Although the two upper stations on the Columbia are in the class 1 range, the majority of the river is well into class 0. A class 0 rating is therefore assigned.

Snake

Discharges at the two USGS stations on the Snake indicate this river is in dilution class 1.

Willamette

Discharge on the Willamette River is, except for Albany, well into the class 1 range.

Stage II

Columbia

Dispersion measurements of radioactive tracer on the Columbia River were reported by Nelson, Perkins and Haushild (1966). With a discharge very close to the 50 percent flow at "Reactor D" in the Hanford area, the dispersion of I^{131} was monitored at several locations downriver. The authors are careful to point out that the Columbia does not meet the requirements of the one-dimensional dispersion model in that cross-sectional area and velocity are not constant across the study reach. At the same time, these measurements provide the best available information on the dispersion of a pollutant in the Columbia at the 50 percent flow level.

The variation in river cross-section due to the control structures causes fluctuations in the stream velocity. Time of travel information presented by Nelson et al. (1966) agrees very closely with the average of the velocities measured at the gaging stations (2.76 fps from Nelson versus 2.36 fps from gaging stations). Velocities at the gaging stations were accordingly used without adjustment.

Using the data presented by Nelson et al. (1966), a value of 4,100 ft^2/sec was computed for D between Finley and Umatilla. Using this value for D and the stream conditions at the gaging stations, the following amounts of spill are required to produce a 1,000 ppm concentration 25 miles downstream. Dispersion coefficient values predicted by equation (3.22) (Fukuoka and Sayre) also are given for comparison.

Mile	D (ft^2/sec)	Tons	W/h	D (Fukuoka/Sayre)
331	4,100	115,750	82.2	3,450
292	4,100	75,500	101.8	7,420
189	4,100	140,300	26.2	1,320
106	4,100	181,300	149.4	11,100
				(average: 5,830)

Willamette

No dispersion measurements were available, so the method of Fukuoka and Sayre was employed to estimate D. From navigation charts, r_c and L were determined to be 4,511 and 11,894 feet, respectively. The following spill sizes required to produce 1,000 ppm 25 miles downstream were then calculated.

Miles (from Portland)	D(ft^2/sec)	W/h	Tons
119	1,000	40.3	5,910
84	2,450	57.2	7,005
37	419	24.2	13,223

Snake

The dispersion coefficient prediction method of Fukuoka and Sayre was employed in the absence of dye measurements. From navigation charts, r_c and L were determined to be 5,906 and 15,060 feet, respectively. The following spill sizes required to produce 1,000 ppm 25 miles downstream were then calculated with the data at the two gaging stations.

Mile	D	w/h	Tons
140	3,150	37.6	9,990
10	7,330	138.2	24,120

Columbia, Snake and Willamette Rivers

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Columbia River near Priests Rapids Dam	90%	58,000	22,500	2.6	1,110	
12472800 River Mile (390)	50%	93,860	27,000	3.5	1,150	
Columbia River near Pasco, WA	90%	60,500	59,000	1.1	2,210	
12514000 River Mile 330.8	50%	88,800	60,000	1.5	2,220	
Columbia River below McNary Dam, WA	90%	91,000	46,000	2.0	2,230	
14019200 River Mile 292.0	50%	137,700	52,000	2.6	2,300	
Columbia River at the Dalles, OR	90%	97,000	92,000	1.1		
14105700 River Mile 188.9	50%	145,000	93,000	1.5	1,560	
Columbia River at Vancouver, WA	90%	117,400	93,800	1.2	3,800	
14144700 River Mile 106.5	50%	304,000	102,000	2.9	3,900	
Snake River below Ice Harbor Dam, WA	90%	21,800	11,700	1.8	1,310	
13353000 River Mile 9.7	50%	37,450	13,000	2.9	1,340	
Snake River near Clarkston, WA	90%	21,800	7,600	2.8	560	
13343500 River Mile 140.0	50%	35,160	9,400	3.8	595	
Willamette River at Albany, OR	90%	4,850	4,700	0.8	460	170.2
14174000 River Mile 119.3	50%	9,715	6,200	1.5	500	172.9

Columbia, Snake and Willamette Rivers, continued

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Willamette River at Salem, OR	90%	6,410	3,800	1.6	425	111.2
14191000	50%	16,030	6,300	2.7	600	114.3
River Mile 84.2						
Willamette River at Wilsonville, OR	90%	6,605	16,000	0.4	650	54.1
14198000	50%	17,460	17,500	1.0	650	56.5
River Mile 37.4						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						

Connecticut River Estuary

The Connecticut River is navigable from its mouth to Hartford, Connecticut, approximately 45 miles.

Stage I

The available dilution volume appears to decrease gradually throughout its length, alternating from class 1 to class 2. The major portion of the distance, as well as the 50% exceedence freshwater discharge at Thompsonville, indicate a class 1 dilution capacity rating.

Stage II

At the lower end of the estuary, the tidal excursion is 5.1 miles and the amount of spill required to produce 1,000 ppm is 23,100 tons. At mile 19, this changes to 3.1 miles and 6,200 tons while at Hartford, Connecticut, the excursion is 2.4 miles and a 4,270 ton spill is required.

Connecticut River and Estuary

LOCATION	AREA ft^2	TIDE VEL fld	ebb	Q cfs
RR Drawbridge, Mile 2	30,000	1.5	1.5	36,300
Mile 9	24,000	1.1	1.4	24,200
Mile 19	13,500	.8	1.0	9,800
Portland	19,800	.9	1.0	15,160
Rocky Hill	13,200	.6	.8	7,500
Hartford	12,000	.1	.7	6,700
50% discharge at Thompsonville				10,000

Cumberland River

Head of navigation on the Cumberland is above Celina, Tennessee, 385 miles above its mouth at Smithland, Kentucky.

Project depth for the waterway is 9 feet. Navigation depth is maintained by four dams along the river's length.

The stream gaging station information is presented in the accompanying table. Several stations are located in the tailwaters of control structures and are, therefore, not representative of the majority of the river reach under consideration. These stations, indicated with an asterisk were adjusted by the method indicated in Chapter VI.

Stage I

With the exception of the Celina station, the Cumberland River is in the class 1 range of dilution volume (10,000 - 100,000 cfs).

Stage II

No dispersion coefficient measurements were available for the Cumberland. Using the prediction relation of Fukuoka and Sayre and the adjusted stream gaging values, with n = .03, the following spills required to produce 1000 ppm concentration 25 miles downriver were obtained.

Mile	D	w/h	Tons
381	2,780	50.6	3,150
308	975	25.9	5,680
212	1,096	47.3	11,140
149	1,398	62.8	14,300
89	622	21.4	12,830
30	1,455	40.0	17,080

Cumberland River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Celina, Tenn. 03417500	90%	2,160	960	2.1	355	492.0
River Mile 380.0	50%	8,630	2,850	3.1	380	497.0
Carthage, Tenn. 03425000	90%	3,370	5,200	0.7	390	444.7
River Mile 308.2	50%	12,810	6,800	1.9	420	447.7
Dam 3 near Old Hickory, Tenn.	90%	3,400	4,000	0.8	360	386.0
03426500 River Mile 212.1	50%	13,200	5,500	2.6	370	389.5
Cheatham Dam, Tenn* 03435000	90%	5,410	6,000	0.9	480	355.6
River Mile 148.7	50%	15,160	7,200	2.1	520	358.2
Dover, Tenn. 03437000	90%	4,400	10,900	0.4	495	335.2
River Mile 88.8	50%	17,075	11,800	1.5	500	336.3
Smithland, Kentucky* 03438220	90%	6,420	5,200	1.25	400	302.5
River Mile 30.5	50%	17,170	6,500	2.7	450	305.4
	90%					
River Mile	50%					
*Dam 3 near Old Hickory	90%					
River Mile 212.1	50%	13,200	(10,400)	(1.3)	(700)	

Cumberland River, continued

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
*Cheatham Dam	90%					
	50%	15,160	(11,820)	(1.3)	(860)	
River Mile						
*Smithland, Ky.	90%					
	50%	17,170	(8,400)	(2.0)	(580)	
River Mile 30.5						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						

Delaware River and Estuary

The Delaware is navigable from its mouth at Cape Henlopen to Trenton, NJ, a distance of 132 miles. The mean tidal range of the navigable reach ranges from 4.1 feet at the mouth to 6.8 feet at Trenton.

The median freshwater discharge at Trenton (estimated by 70% of the average discharge) is approximately 8,400 cfs. During the summer months, this discharge may drop to as low as 1,200 cfs. Comparing this to the average tidal discharges taken from the tidal current tables indicates that tidal motion is the dominant mixing method in the bay and estuary.

Stage I

From the mouth to Philadelphia, the discharge information indicates a class 0 rating. Above Philadelphia, the rating is 1.

Stage II

The following spill amounts necessary to produce 1,000 ppm in the tidal excursion were computed.

Location	Mile	Tidal Excursion (miles)	Tons
Arnold Pt.	40	14.1	448,000
C and D Canal	59	14.3	226,000
Deepwater Pt.	70	19.1	194,000
Marcus Hook	78	11.2	112,000
Philadelphia	90	14.3	84,300

Location	Mile	Tidal Excursion (miles)	Tons
Fisher Pt.	95	10.6	49,300
Bristol	119	9.9	22,500
Whitehill	126	4.9	13,000

Delaware River and Estuary

LOCATION	AREA ft^2	TIDE VEL fld	ebb	Q cfs
Arnold Pt.	425,000	2.0	2.1	703,000
C and D Canal ent.	210,000	2.0	2.2	355,000
Deepwater Pt.	135,000	3.0	2.6	304,000
Marcus Hook	132,000	1.7	1.6	175,000
Gloucester	60,000	2.2	2.0	101,000
Fisher Pt.	62,000	1.4	1.7	77,400
Bristol	27,600	1.3	1.6	32,200
Whitehill	35,000	-	.9	13,000

Green and Barren Rivers

The Barren River is navigable from Bowling Green, Kentucky to its confluence with the Green River at mile 145.5 of the Green River, a distance of 30 miles. The Green River is navigable from mile 168 to its confluence with the Ohio at Ohio mile 784.

Project depth is 9.0 feet up to mile 103.0 where the depth is maintained at 5.5 feet. Depths are maintained by a series of five dams along the Green and one dam along the Barren. Four USGS gaging stations are located along the river system.

Stage I

The 50% discharge in the Green and Barren Rivers fall into class 2 (10,000 - 1,000 cfs).

Stage II

As no dispersion measurements are available, the prediction method of Fukuoka and Sayre (1973) is used. From the Corps of Engineers navigation charts, average bend radius was estimated at 1,410 feet and average bend length at 4,000 feet. The following values for D, W/h and tons required to produce 1000 ppm were then calculated.

Mile	D ft^2/sec	W/h	Tons
149.1	157	26.2	1,385
100.1	242.	36.6	1,467
63.2	96.4	19.0	1,886

Green and Barren Rivers

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Barren R. at Bowling Green, Ky. 03314500 River Mile 37.6	90%	230	Above head of navigation			
	50%	1,280				
Green R. at Woodbury 03315500 River Mile 149.1	90%	720	1,800	.4	255	382.1
	50%	3,070	3,000	1.05	280	384.9
Green at Paradise, Ky. 03316500 River Mile 100.1	90%	760	2,200	.34	310	364.0
	50%	3,375	2,800	1.2	320	365.7
Green at Calhoun, Ky. 03320000 River Mile 63.2	90%	1,100	4,200	.25	280	363.75
	50%	3,960	4,800	.85	285	364.95
River Mile	90%					
	50%					
River Mile	90%					
	50%					
River Mile	90%					
	50%					
River Mile	90%					
	50%					

Gulf Intracoastal Waterway--West Florida Section

The intracoastal waterway is navigable on the west coast of Florida from Tampa to its junction with the Caloosahatchee River Waterway, a distance of 95 miles. Controlling depth for the waterway is 9 feet.

The first 35 miles (mile 95 to mile 60 with miles numbered from San Carlos Bay, Florida) are through open bay section. Mile 60 to 50 is primarily a land-cut section through a shallow bay which opens significantly by mile 50. From there, the waterway proceeds through Lemon Bay and Gasparilla Sound to Charlotte Harbor. From there it passes through Pine Island Sound, San Carlos Bay and into the Calooshatchee River at mile 0.

Stage I

This section is primarily in open bays, indicating a dilution class 0. Miles 60 to 50, however, are either land-cut or dredged through very small bay areas. Using a diurnal tide range at Venice of 2.1 feet, width of 300 feet and total length of 10 miles, and conservatively assuming a uniform tidal range gives a discharge at mile 56 (Venice) of 740 cfs. The average over the land-cut reach is then 370 cfs indicating a class 3.

Stage II

In the land cut section (mile 60-50) the spill volume required to produce 1000 ppm is the tidal excursion calculated to be 2.6 miles long is 940 tons. No calculations were made on the open bay sections.

Gulf Intracoastal Waterway--Florida Panhandle Section

The Gulf Intracoastal Waterway extends eastward from Alabama at mile 167 from Harvey Lock, Louisiana to its termination at mile 380. Connection with the rest of the GIW network is authorized but not yet completed.

From mile 167 to mile 254, the waterway runs through Perdido, Pensacola and Choctawhatchee Bays. Miles 254 to 273 is a land cut section. Miles 273 to 313 run through West, St. Andrew and East Bays. From mile 313, the waterway is cut through several small creeks and enters Lake Wimico at mile 335. Leaving Lake Wimico at mile 341, it follows the Jackson River to its mouth at mile 351. From there, the GIW follows Apalachicola Bay to the end of the maintained portion of the GIW at mile 380.

Stage I

The following dilution capacity ratings were assigned.

Mile 167-254: Class 0. Open bay section

Mile 254-273: Class 3. Discharge at mouths of land cut section calculated to be 245 cfs with diurnal tidal range of 1.1 feet. The average is 122 cfs or barely in class 3 range.

Mile 313-335: Class 3. Discharge at mile 313 based on surface area up to Lake Wimico is 520 cfs. Average over the reach is taken as 260 cfs.

Mile 335-350: Class 2. Using the surface area of the lake as well as the surface area of the Jackson River, and a tidal range of 1.1 feet indicates a discharge at the mouth of the Jackson River of 4700 cfs. At mile 341, the upper end of the Jackson River, the discharge is still indicated to be 1600 cfs assuming no freshwater flow into the lake.

Mile 350-380: Class 0. Open bay section.

Stage II

No spill volumes were calculated for the open bay sections. On the land cut set sections, where spill dilution is contained by action of the waterway boundaries, the following spill volumes required to produce 1000 ppm in the tidal excursion were calculated.

At mile 254 and mile 273, the mouths of a 19 mile long land cut section, tidal induced discharge was 245 cfs. This gives a tidal excursion of 1.1 miles and requires 311 tons to produce 1000 ppm.

Mile 313 is the mouth of the 22 mile long land cut section which enters Lake Wimico. The tidal induced discharge at this point, assuming no freshwater discharge from the lake is 520 cfs. The tidal excursion is then 2.2 miles and 660 tons are required to produce 1000 ppm in the excursion.

In Jackson River, mile 341 to mile 350, the surface area of Lake Wimico (5.2 mi^2) produces a greater tidal discharge through the Jackson River. At the mouth, a 4700 cfs discharge was calculated using no freshwater discharge and a uniform tidal range of 1 foot throughout the river and lake. This discharge in the Jackson corresponds to a tidal excursion of 2.2 miles with 9,970 tons required to produce 1000 ppm.

At the upper end of the river, a 1600 cfs discharge was computed using a tidal range of .5 feet. This corresponds to a .91 mile tidal excursion and a 2,030 tons spill required to produce 1000 ppm.

Gulf Intracoastal Waterway--Alabama Coast

The Gulf Intracoastal Waterway runs through Alabama from mile 113 to mile 166. Of this distance, only 10 miles (150-160) are land cut canal sections while the rest is through Mississippi Sound and Alabama Bay.

Stage I

The dilution capacity rating for the open bay portions of the GIW is class 0.

In the 10 mile stretch of land cut waterway, assuming equal tide levels on both ends of the cut, a tide induced discharge of 180 cfs at each mouth of the land cut section was computed.

The average Q for this reach is then 90 cfs indicating a class 4 rating (<100 cfs).

Stage II

No spill volume was computed for the open bay section. In the canal land cut section, the diurnal tidal excursion at the mouths is computed to be approximately .8 miles. The spill size required to produce 1000 ppm in the tidal excursion is 233 tons.

Gulf Intracoastal Waterway--Mississippi Coast

The Gulf Intracoastal Waterway along the State of Mississippi runs from mile 37 (from Harvey Lock, Louisiana) to mile 112. The entire distance of this reach is in Mississippi Sound, a shallow bay protected by barrier islands.

Diurnal tidal range along the Mississippi coast is 1.5 to 1.7 feet. Although limited information is available on flushing times for these bays, it is safe to assume that spill dilution capacity of the bays is large compared to other more restricted waterways.

Stage I

A dilution class of 0 is assigned.

Stage II

No spill tonnages were computed for the tidal excursion of Mississippi Sound.

Gulf Intracoastal Waterway--Texas Coast

The Gulf Intracoastal Waterway (GIW) was completed to its present dimensions (12 feet deep by 125 feet bottom width) in 1949 all the way to the Mexican border. It is dredged through flat coastal land and shallow bays for its entire length in Texas.

Starting from the Sabine River, at mile 265 (measured from its junction with the Mississippi River), the GIW proceeds inland to Bolivar where it joins Lower Galveston Bay at mile 349. From there it winds through a channel dredged in West Galveston Bay, and goes inland again from mile 378 to Brazosport Harbor at mile 395, and inland again, crossing the Brazos River at mile 401 and the Colorado River and lock at mile 442. The GIW then proceeds through shallow bays, principally Laguna Madre, with small inland cuts, to its termination at Port Brownsville at mile 665.

Of the total length of the GIW through Texas, approximately 173 miles or 44% of the waterway is inland cuts while the remainder is through bays protected by barrier islands.

<u>Stage I</u>

Mile 265 - 349: Class 3. Mean diurnal tide ranges are 1.4 and 1.3 feet, respectively, at the ends of the sections. With a length of 70 miles and a surface width of 300 feet, the flows at the ends, assuming a uniform tide along the reach, are 1,680 cfs. The average discharge for this section is then 840 cfs.

Mile 349 - 363: Class 1. This is an open bay section of the

GIW with spill dilution limited primarily by slow flushing of some of the bay sections.

Mile 363 - 455: Class 3. This land-cut reach has openings to the ocean at mile 376, 382, 395, 401, 405, 441, and 450. Using a mean tide range of 2.0 feet, mean tidal discharge from this section ranges from 105 to 740 cfs.

Mile 455 - 655: Class 1. This is primarily bay sections with short land-cut reaches. Although dilution volume is quite large in these bay sections, it is limited somewhat by spoil banks in the bays and the relatively long flushing times. Dilution rating was therefore adjusted to a class 1.

Stage II

The following spill volumes necessary to produce 1,000 ppm in the tidal excursion were computed.

Mile	Tidal Excursion	Tons
265	5.7	2,130
349	5.7	2,130
363	4.9	880
376	3.7	470
382	1.8	210
395	2.2	180
401	1.7	160
405	1.8	180
441	4.8	820
450	2.1	240

Houston Ship Channel

The Houston Ship Channel extends for 52 miles from Galveston to Houston, Texas. The lower 28 miles are through a dredged channel in Galveston Bay while the upper 24 miles are dredged from Buffalo Bayou. Traffic in OHM is very heavy on the HSC.

The channel is maintained at 40 feet for a width of 400 feet. Mean diurnal tidal range at Morgans Point (mile 28) is 1 foot. Freshwater flow is small in the upper HSC, with the 90% and 50% flows at mile 38 being approximately 100 and 400 cfs respectively. Maximum tidal current velocities have been reported between .2 and 2.0 ft/sec at mile 38.

Stage I

Mile 0 - 28: Class 0. Relatively open bay.

Mile 28 - 52: Class 2. At the lower flow conditions, numerical model studies have indicated a maximum tidal velocity of .24 fps at mile 34, with a steady decrease upstream. Integrating this velocity over a diurnal tidal cycle yields an average Q of 3300 cfs at mile 34. Tidal cubature yields an average Q of 2060 cfs. The discharge averaged over the length of the upper HSC is then barely in the class 2 range.

Stage II

Using tidal cubature to estimate the flows in the upper channel yields the following spill amounts to produce 1000 ppm in the tidal excursion.

Mile	Area	Tidal Excursion	Tons
52	16,000	.2	450
38	22,000	.8	2,575
28	24,000	1.0	3,490

Hudson River Below Troy Lock and Dam

The Hudson River is a tidal estuary below Troy, with the range in tide nearly constant along its length. Tidal currents are generally strong along its length, although cross-sectional area is sharply reduced in the upper reaches. Tidal currents are strongly semidiurnal, with the tide lagging the downstream tide progressively further with distance upriver.

<u>Stage I</u>

Available dilution water exceeds 100,000 cfs to above Kingston Pt. (approximately mile 90), where it drops rapidly to approximately that of the Hudson River freshwater inflow at Troy. Class 0 is indicated from the mouth to mile 90, and class 1 from there to Troy.

<u>Stage II</u>

At the lower end of the Hudson River, the average tidal excursion is computed to be 6.5 miles and 158,000 metric tons spilled would be required to produce 1000 ppm in the tidal excursion. By West Point these figures have dropped to 3.6 miles and 108,600 tons respectively and at Kingston, these values are 4.95 miles and 80,900 tons. Above Kingston, the tidal excursion remains approximately 5.8 miles but the amount of spill required for 1000 ppm concentration drops to 25,000 tons. At Albany, tidal excursion is 1.9 miles and only 4,400 tons are required to produce the same concentration.

Hudson River below Troy, NY

LOCATION	AREA ft^2	TIDE VEL fld	ebb	Q cfs
The Battery	162,000	1.5	2.3	248,000
George Washington Bridge	142,500	1.6	2.2	218,200
Tarryton	165,000	1.1	1.5	172,800
Peekskill	157,500	.8	1.2	127,000
West Point	201,600	1.0	1.1	170,600
Newburgh	156,000	.9	1.1	125,700
Poughkeepsie	117,000	1.1	1.2	108,500
Kingston Pt.	109,000	1.3	1.6	127,000
Catskill	27,000	1.5	2.0	38,000
Coxsackie	29,400	1.6	1.8	40,200
Albany	7,000	.3	.8	3,100
Troy	6,000		.7	3,400

Illinois River

The Illinois River has its source in Chicago Harbor and the Calumet-Sag Channel systems flowing from Lake Michigan. From the Chicago Sanitary and Ship Canal, the Illinois River flows 354 miles to its confluence with the Mississippi River at Grafton, Illinois.

Flow is controlled by a series of 7 dams which also serve as power generation facilities. Project depth of the waterway is 9 feet and width is 200 feet.

Stage I

From the 50% discharges, a class 2 (10,000 - 1,000 cfs) rating is indicated. The lower end of the river does, however, exceed this value, while portions of the Chicago and Calumet Canals are below this range.

Stage II

Dispersion coefficients measured in the Chicago Sanitary and Ship Canal by Thomas (cited in Fischer, 1973) were $D/hu^* = 20.0$, indicating the absence of strong velocity shears and dead zones. This would, of course, be expected in the rock-cut sections of the Canal, where w/h ranges from 18.0 to 6.0.

In the river sections below Lockport, the river has very slight, long radius meanders. In this situation, the predictive relation of Fukuoka and Sayre must be used with caution because a straight channel would result in an infinite D. The results obtained did, however, (r_c = 6990', L = 13,300') fit well with data presented in Figure 3.1. The somewhat higher values of D resulting from the gentle meanders are to some extent realistic because of the large amount of shallow dead zone areas outside the navigation channel indicated on the navigation charts. With n = .02, the following spill sizes were computed.

Chicago Sanitary and Ship Canal

Mile	D	w/h	Tons
291	22.3	6.0	715

The rest of the Illinois River was evaluated, with n = .03, using the method of Fukuoka and Sayre.

Mile	D	w/h	Tons
246.6	2,138	51.4	11,940
145.3	3,806	68.0	13,850
70.8	3,526	59.5	16,190

Illinois Waterway

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Lockport, Ill. 05537000	90%					
River Mile 291.0	50%	(3,950)*	(4,100)	(1.0)	160	
Marseilles, Ill.+ 05543500	90%	4,330	1,500	3.0	610	464.5
River Mile 246.6	50%	6,850	1,860	3.7	610	465.2
Kingston Mines, Ill. 05568500	90%	4,665	4,800	.85	650	430.7
River Mile 145.3	50%	9,460	7,200	1.4	700	433.8
Meredosia, Ill. 05585500	90%	5,970	7,200	.82	700	420.4
River Mile	50%	13,800	9,200	1.55	740	423.7
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%	6,850	(7,000)	(1.0)	600	
	90%					
River Mile	50%					

* 50% Q estimated by multiplying mean flow, 5,731 cfs, by the average ratio of 50% flow to mean flow (0.69). Cross-sectional area taken from navigation chart.

\+ The Marseilles Station is in rapids that are bypassed by the canal. Cross-sectional area estimated from navigation charts yields these values which were used for this station.

James River Estuary

The James River is navigable to Richmond, VA, a distance of 87 miles above Newport News, VA. Controlling depth for the waterway is 25 feet to Richmond.

Stage I

Tidal and freshwater flows presented in the accompanying table indicate a broad range depending on position in the estuary. Although the flow is less near Richmond and greater in the lower bay, a class 1 dilution capacity rating is assigned because it is fairly representative of the estuary.

Stage II

Using the flows obtained from tidal current information, the following spill size required to produce 1,000 ppm in the tidal excursion were calculated:

Mile (above Newport News)	Tidal Excursion (miles)	Tons
0	3.7	122,200
17	3.9	152,800
30	4.1	79,500
44	4.2	45,400
68	3.7	13,100
87	3.4	5,350

James River Estuary

LOCATION	AREA ft^2	TIDE VEL fld	ebb	Q cfs
Newport News	218,000	1.0	1.2	191,800
Deepwater Shoal	260,000	1.2	.9	240,000
Church Pt.	130,000	1.1	1.3	124,800
Brandon Pt.	71,300	1.2	1.3	71,300
Bermuda Hundred	23,400	.9	1.3	20,600
Rocketts (Richmond)	10,500	-	1.0	8,400
Median freshwater discharge at Richmond				3,570

Kanawha River

The Kanawha River is navigable from Deepwater, West Virginia, a distance of 91 miles to its confluence with the Ohio River at Point Pleasant, West Virginia. Flow is regulated by three dams along its length. Project depth for the Kanawha is 9 feet.

Stage I

The gaging station information available indicates this river is in class 2 (10,000 - 1,000 cfs).

Stage II

No dispersion coefficient measurements were available for the Kanawha River. Using the method of Fukuoka and Sayre, with r_c and L determined to be 7,030 and 13,450 feet respectively, and n = .03, the following value at the Charleston station was determined.

Mile	D	w/h	Tons
54	2,965	80.6	19,580

Kanawha River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Kanawha Falls, WV 03193000	90%	2,710	above head on navigation			
River Mile (100)	50%	7,460				
Charleston, WV 03198000	90%	3,200	9,200	.35	860	566.0
River Mile 54.3	50%	8,950	9,500	.95	875	566.0
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					

Kennebec River Estuary

The Kennebec River is navigable from its mouth to Augusta, Maine, a distance of 36 miles. Tidal currents are semidiurnal and in general quite strong.

Stage I

In the lower reaches up to Bath, the most probable discharge available for dilution is greater than 100,000 cfs or class 0. Above Bath, tidal influence is lessened until the 50% discharge at Bingham, above Augusta is 3,375 cfs. The dilution class indicated from Bath to Augusta is Class 1.

Stage II

From the mouth to Bath, most probable discharge averaged over the length is 110,000 cfs. Tides are semidiurnal with one half period of 6.24 hours. Average cross-sectional area is 61,000 ft^2. Computing the tidal excursion from this yields 7.7 miles. The amount of material required to produce 1000 ppm concentration is 70,000 tons.

Using navigation charts, waterway surface area was computed for the reach from Bath to Augusta, Maine. Mean tidal range at Bath is 6.4 feet while at Augusta it is 4.1 feet. Using tidal cubature, an average tidal discharge of 16,000 cfs was computed for mile 22 and 5,600 cfs for mile 30. At mile 22, the tidal excursion is 3.8 miles and 10,200 tons are required for 1000 ppm in the tidal excursion. At mile 30, the tidal excursion is 1.9 mile and

2,150 tons of spill are required for an equivalent concentration in the tidal excursion.

Kennebec River & Estuary

LOCATION	AREA ft^2	TIDE fld	VEL ebb	Q cfs
Mile 0	63,000	2.4	2.9	134,500
Mile 3	72,000	1.6	2.3	113,100
Mile 6.5	52,500	2.3	3.4	120,500
Mile 8	52,500	2.6	3.0	118,400
Bath, Mile 10	64,800	1.0	1.5	65,200
Mile 22	18,000			16,000
Mile 30	12,600			5,600
Augusta, Mile 36	3,000			
Bingham, above tidal influence	50% flow			3,375

Kentucky River

The Kentucky River is formed by the junction of the North and Middle Forks east of Beattysville, Kentucky and flows to the Ohio River at Carrollton, Kentucky, for a navigable distance of 258.6 miles. Project depth is 6 feet and width is 100 feet. Flow is regulated by a series of 14 dams with locks, the smallest of which is 38 x 145 feet in size. Six Geological Survey Gaging Stations are located along the Kentucky.

Stage I

All of the flow information at the gaging stations indicate the river is in the class 2 range (1,000 - 10,000 cfs).

Stage II

No dye dispersion information was available. Using the method of Fukuoka and Sayre (1973), with n = .03, and bend radius and length taken from navigation charts to be 2,280 and 6,916 feet respectively, the following dispersion coefficient predictions were obtained. Using these values, the spill sizes required to produce 1000 ppm were calculated.

Mile	D	W/h	Tons
249.2	45.2	17.8	1,562
176.4	96.0	20.6	1,329
139.9	151.2	27.0	1,337
96.2	96.1	20.0	1,588
65.9	242.8	27.0	1,045

Mile	D	w/h	Tons
31.0	173.1	24.5	1,637

Kentucky River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Lock 14 near Heidelberg, Ky.	90%	230	3,250	0.07	245	635.7
03282000 River Mile 249.2	50%	1,090	3,500	0.31	250	636.4
Lock 10 near Winchester, Ky.	90%	320	1,800	0.17	240	567.0
03284000 River Mile 176.4	50%	1,640	2,800	0.58	240	568.0
Lock 8 near Camp Nelson, Ky	90%	360	1,900	0.17	260	530.6
03284500 River Mile 139.9	50%	1,780	2,500	0.72	260	531.8
Lock 6 near Salvisa, Ky.	90%	450	3,200	0.15	240	497.3
03287000 River Mile 96.2	50%	2,025	3,400	0.6	260	498.4
Lock 4 near Frankfort, Ky.	90%	540	1,700	0.32	215	468.5
03287500 River Mile 65.9	50%	2,215	1,950	1.15	230	469.3
Lock 2 near Lockport, Ky.	90%	540	2,500	0.2	270	441.3
03290500 River Mile 31.0	50%	2,730	3,200	0.9	280	442.2
	90%					
River Mile	50%					
	90%					
River Mile	50%					

Upper Mississippi River

The Mississippi River is navigable from Minneapolis, Minnesota to its mouth in the Gulf of Mexico. Mile designations for the upper portion are from the confluence with the Ohio River, a distance of 857.6 miles to the head of navigation.

The waterway has been improved to provide a minimum depth of 9 feet with widths of 200-400 feet by means of a system of 26 dams and locks. The channel is also stabilized by means of dikes and revetments.

Navigation season for the upper Mississippi is 9 months long from the end of March to the first week in December.

Stage I

The upper Mississippi, due to its great length, varies greatly in properties. Dilution class ratings are therefore designated as follows.

Mile 0 to Mile 195 (above confluence with Missouri) class 0
Mile 195 to Mile 812 (above confluence with the St. Croix River) class 1
Mile 812 to Mile 857 (head of navigation) class 2

Stage II

No dispersion measurements were available for the Upper Mississippi River. The prediction method of Fukuoka and Sayre was used, with r_c and L determined from navigation charts to be 8,870 and 21,660 feet respectively. With n = .03, the following values were obtained for the amount of spill required to produce 1,000 ppm concentration 25 miles downstream.

Mile	D	w/h	Tons
865	3,080	69.8	7,540
726	1,660	58.0	30,100
633	5,120	165.8	54,100
512	3,780	89.3	48,300
364	14,600	226.9	51,900
203	4,220	78.5	70,700
178	6,380	61.0	68,170
110	7,340	73.5	66,150
44	9,997	107.5	88,700

Upper Mississippi River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Anoka, Minnesota 05288500	90%	1,750	(3,000)	(.08)	(520)	806.3
River Mile 864.8*	50%	5,470	4,200	1.3	615	808.0
St. Paul Minnesota 05331000	90%	2,400				
River Mile 839.3	50%	6,000				
Prescott, WI 05344500	90%	4,460				
River Mile 811.4	50%	9,780				
Winnona, Minnesota 05378500	90%	9,185	(15,000)	(0.6)	(1,010)	685.1
River Mile 725.7	50%	16,714	19,000	0.9	1,050	685.2
McGregor, IA 05389500	90%	12,910	19,500	.65	1,800	605.3
River Mile 633.4	50%	23,340	21,500	1.1	1,890	612.1
Clinton, IA 054205	90%	18,500	22,500	.82	1,460	562.7
River Mile 511.8	50%	35,800	25,200	1.4	1,500	566.9
Keokuk, IA 05474500	90%					
River Mile 364.2	50%	(43,095)[1]	18,000	2.4	2,020	488.1
Alton, Ill. 05587500	90%	31,800	(30,000)	(1.1)	1,740	
River Mile 202.7	50%	69,300	39,000	1.75	1,750	401.7

Upper Mississippi River (cont.)

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
St. Louis, MO	90%	66,270				
07010000 River Mile (178)	50%	138,000	42,000	3.3	1,600	
Chester, IL	90%	65,670				
07020500 River Mile 109.9	50%	130,555	38,000	3.3	1,670	
Thebes, IL	90%	65,000				
07022000 River Mile	50%	135,650	43,000	3.2	2,150	312.3
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					

Lower Mississippi River

The lower Mississippi extends from the confluence of the Ohio and upper Mississippi Rivers at Cairo, IL to Head of Passes, LA, a distance of 956 miles. Mileages are measured from Head of Passes (mile 0).

The lower Mississippi is controlled for navigation by dikes and revetments instead of dams. A number of the meanders have been cut off to reduce navigation distances and stabilize the channel.

From the mouth to Baton Rouge (233.4) the navigable channel is maintained at 40 feet deep and 500 feet wide. From Baton Rouge to Cairo, IL, the channel is 12 feet deep and 300 feet wide.

Stage I

All of the stations along the Mississippi indicate a median flow in the class 0 (greater than 100,000 cfs) range.

Stage II

Several dye dispersion time of travel studies have been conducted on the Mississippi between Baton Rouge and New Orleans (Stewart, 1967; Martens, 1974). From these, a D value of 2,500 ft^2/sec was determined for the median flow. The dispersion coefficient was also estimated by the Fukuoka and Sayre method and found to be similar but somewhat higher (approximately 4,000 ft^2/sec). The value obtained by dye tests on the lower reaches was used on the entire river because the flow and characteristics do not change greatly over the length of the river.

Using the measured value for D, the following amounts of solute required to produce 1,000 ppm, 25 miles downstream, were calculated:

Mile	D	W/h	Tons
731.5	2,500	42.1	79,400
663.3	2,500	78.2	82,000
554.3	2,500	72.2	101,300
430.4	2,500	53.3	81,700
230.0	2,500	81.0	126,760

Lower Mississippi River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Memphis, TN	90%	160,000	66,000	2.2	1,850	183.2
07032000 River Mile 731.5*	50%	356,000	85,000	3.9	1,890	193.3
Helena, AR	90%	175,000	62,000	2.7	2,350	146.7
07047970 River Mile 663.3	50%	380,000	90,000	4.1	2,650	158.2
Arkansas City, AR	90%	195,950	95,000	2.1	3,250	99.2
07265450 River Mile 554.3	50%	430,800	125,000	3.4	3,400	109.8
Vicksburg, Miss.	90%	195,000	70,000	(2.9)	1,700	50.8
Corp of Eng. Gage River Mile 430.4	50%	445,000	95,000	4.6	2,250	60.8
Baton Rouge, LA	90%	180,000	92,000	1.9	2,830	
Corp of Eng. Gage River Mile 230.0	50%	335,000	117,000	2.9	3,080	10.6
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					

* Miles are from Head of Passes, LA

Missouri River

The Missouri River is navigable for 732 miles from Sioux City, Iowa to its confluence with the Mississippi 17 miles above St. Louis, Missouri. River miles are counted from the confluence upstream. Project depth is 9 feet while project width is 300 feet. The Missouri is controlled without the use of locks or dams by means of dikes and revetments. Dikes are used to control the channel by restricting flow on the inside or convex side of bends, thus making the river more narrow and deep. Revetments are used on the outside or concave sides of the bends to protect the bank from erosion.

Navigation season is from April through November. Nine USGS gaging stations are spaced along the river.

Stage I

Discharge for entire length is in the class 1 (100,000 - 10,000 cfs) range.

Stage II

Using the dispersion coefficients measured by Yotsukura, Fischer and Sayre (1970) of 16,000 ft^2/sec, the amount of toxicant required to produce 1000 ppm concentration of a solute 25 miles downstream is:

Mile	Tons
723.3	21,953
615.9	11,983

Mile	Tons
562.6	56,365
498.0	18,035
448.2	44,173
366.1	24,350
293.4	23,700
196.6	24,255
97.9	36,300

Missouri River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Sioux City, IA 06486000	90%	8,720	3,500	2.5	400	1089.0
River Mile 723.3	50%	27,300	8,500	3.2	700	1095.0
Omaha, NB 06610000	90%	9,390	2,800	2.8	450	961.4
River Mile 615.9	50%	28,630	5,800	5.0	580	963.7
Nebraska City, NB 06807000	90%	15,340	4,900	3.1	500	908.5
River Mile 562.6	50%	33,725	8,000	4.3	660	912.5
Rulo, NB 06813500	90%	14,400	(4,500)	(3.0)	(360)	840.2
River Mile 498.0	50%	34,950	8,000	4.2	610	845.2
St. Joseph, MO 06818000	90%	12,575	(9,600)	(1.3)	500	
River Mile 448.2	50%	32,760	14,550	(2.3)	800	798.2
Kansas City, MO 06893000	90%	14,550	(5,400)	(2.8)	(600)	(717.8)
River Mile 366.1	50%	37,270	10,000	3.6	910	720.8
Waverly, MO 06895500	90%	14,800	(7,000)	(2.1)	(550)	651.5
River Mile 293.4	50%	36,870	10,000	3.6	910	720.8
Boonville, MO 06909000	90%	16,930	(8,500)	(2.3)	(720)	(569.0)
River Mile 196.6	50%	40,640	10,500	4.0	960	572.9

Missouri River, continued

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Hermann, MO 06934500	90%	21,770	(9,500)	(2.3)	(850)	(485.4)
	50%	49,250	14,500	3.4	1,100	489.1
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						

Monongahela River

The Monongahela River is formed at Fairmont WV by the confluence of the Tygart River and the West Fork River. It flows northwesterly for 129 miles to its confluence with the Allegheny River at Pittsburgh. Project depth is 9 feet and width is 300 feet. There are ten dams along its length.

Stage I

The 50% duration flow for the entire river is in the class 2 range (10,000 - 1,000 cfs).

Stage II

As no dispersion measurements were available, the prediction method of Fukuoka and Sayre (1973) was employed. From navigation charts, bend radius and bend length were determined as 5,650 and 14,500 feet respectively. Calculating u* by the method of Chow (1959) and taking Mannings n as .025, the following values were obtained for the amount of spill in metric tons required to produce 1000 ppm concentration 25 miles downriver.

Mile	D	W/h	Tons
124.2	404	42.5	4,820
85.2	574	65.0	8,560
41.7	617	57.6	8,710
11.2	691	65.6	11,200

Monongahela River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Lock 15, Hoult, W.Va. 03062000	90%	520	2,070	.24	345	857.1
	50%	2,436	4,770	.54	450	858.1
River Mile 124.2						
Greensboro, Pa. 03072500	90%	906	6,200	0.1	650	779.0
	50%	4,670	7,500	0.6	700	780.7
River Mile 85.2						
Charleroi, Pa. 03075000	90%	1,030	6,300	0.15	650	745.9
	50%	5,940	8,500	0.8	700	747.8
River Mile 41.7						
Braddock, Pa. 03085000	90%	2,020	9,950	0.2	800	719.8
	50%	6,945	10,000	0.75	810	720.9
River Mile 11.2						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						

New York Barge Canal System

The barge canal system consists of 524 miles of waterway with 210 miles in river channels, 220 miles of man-made canals, and 94 miles of lake passages. Project depth is a minimum of 12 feet on the system with 14 feet available on the Erie Canal from Three Rivers to Waterford, NY. The main artery is the Erie Canal, supplemented by the Oswego and Champlain Canals.

The Erie Canal from Three Rivers to the confluence with the Hudson at Waterford is made up of sections of the Mohawk River, combined with man-made canal sections. Flow is regulated by a series of dams which also serve as power generation facilities. Pool levels are maintained relatively constant, but power production causes significant daily and weekly fluctuations. A similar situation exists on the Hudson River above Troy Lock and Dam which forms part of the Champlain Canal. To the west, the Erie Canal from Lake Erie (Tonawanda) to Three Rivers is mainly man-made canal sections with less power generation use.

Flow measurements are made only by the USGS at several points along the system. In addition, time of travel measurements, providing dye dispersion information, were available for the Mohawk and Hudson Rivers (Shindel, 1969a,b).

Stage I

The majority of the NY State barge canal system has a discharge in the class 2 range. The western end of the Erie Canal, from Montezuma westward, however, is in the class 3 range as indicated by the 50% discharge at Lock 30 of 290 cfs.

Stage II

Western Erie Canal: This section has low flow and is primarily man-made canal. As such, D can be compared with measured values in other similar waterways such as the Chicago Sanitary and Ship Canal where W/h=6.0 and D/hu*=20.0 (Fischer, 1973). With n=0.022, u*=0.014 and D=3.3 ft^2/sec.

For the Hudson River above Troy, NY, D was measured from time of travel data of Shindel (1969b) as 460 ft^2/sec at two flows differing by a factor of two. Shindel (1969a) also made dye studies on the Mohawk River. For a river- canal reach from Route 50 to Vischer Ferry, D was measured at 43 ft^2/sec at a flow approximately at the 90% exceedence level. Since the canal is held at the same depth of flow for the 50% level, the same D was used. The following spill volumes were computed at the gaged points in the NY State Barge Canal system.

Station	D	W/h	Tons
Erie Canal, Lock 30	3.3	7.9	154
Oswego Canal, Lock 7	38.2	8.6	226
Seneca River at Baldwinsville	43.0	16.6	612
Mohawk River at Little Falls	43.0	8.6	391

Station	D	W/h	Tons
Mohawk River at Cohoes	43.0	31.5	2,400
Hudson River at Green Island	460.0	84.8	9,140

New York State Barge Canal System

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Erie Barge Canal Lock 30, Macedon 04219000	90%	8.6	(1,140)	.007	95	446.0
River Mile	50%	290	(1,140)	.25	95	
Oswego Canal at Lock 7, Oswego 04249000	90%	1,650	(1,680)	.98	120	255.8
River Mile	50%	4,830	(1,680)	2.87	120	
Seneca River at Baldwinsville 04237500	90%	1,025	(2,400)	.43	200	363.0
River Mile	50%	2,187	(2,400)	.91	200	
Mohawk River at Little Falls 01347000	90%	832	(1,680)	.5	120	322.2
River Mile 75.5	50%	1,820	(1,680)	1.08	120	
Mohawk River at Cohoes 01357500	90%	1,220	(6,500)	.18	450	49.1
River Mile	50%	3,215	(6,500)	.43	450	
Hudson River at Green Island 01358000	90%	3,785	9,600	.42	880	15.9
River Mile	50%	8,147	10,200	.78	930	16.7
	90%					
River Mile	50%					
	90%					
River Mile	50%					

Ohio River

The Ohio is formed by the confluence of the Monogahela and Allegheny Rivers and flows 981 miles to its confluence with the Mississippi at Cairo, Illinois. Navigation is maintained with a series of locks and dams along its length. Project depth is 9 feet while width is 400 to 600 feet. Originally 43 low dams were used to regulate flows. These, however, are being replaced by 19 higher dams that form deeper navigation pools and require less lockage.

Gaging stations are maintained by USGS at 12 points along the river. Data from these stations are summarized in the accompanying table.

Stage I

All except the last station on the Ohio show 50% discharges in the class 1 range (100,000 - 10,000 cfs).

Stage II

At low flow conditions, the Ohio is maintained at navigable depths by dams forming a series of nearly level pools.

No dispersion measurements were located for the Ohio River, so it was necessary to use prediction relations to obtain D. Since at lower flow conditions, the Ohio becomes a series of nearly level pools with very slight slope (Steacy, 1961), the prediction relation of Fukoka and Sayre (1973) was employed. Average bend radius, r_c, was found from navigation charts to be 8,860 while average bend length, L, was found to be 16,780 feet.

An inspection of the location of the gaging stations on the navigation charts indicated that most of the stations were not located in close proximity to control structures. The Louisville, Kentucky station, however, was located directly downstream of McAlpine Dam where the flow was relatively rapid compared to the rest of the reach further downstream. The Louisville values were therefore adjusted by an appropriate percentage when computing the spill volume in that location.

Using n = .03, the following values for D and volume of spill required to produce a 1000 ppm concentration 25 miles downstream were computed.

River Mile (from Pittsburg)	D ft^2/sec	W/h	Tons
11.8	2,860	60	48,300
155.0	6,580	57.6	38,500
184.4	2,640	59.4	65,900
311.6	3,670	55.2	65,400
322.5	2,150	31.4	67,500
408.3	7,350	80.0	78,800
470.5	3,340	36.5	57,800
607.3	7,620	110.0	54,500
729.3	7,260	78.9	85,500
903.1	7,280	65.7	96,500
944.0	5,910	100.8	194,000

Ohio River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Swickley, PA 0308600	90%	5,950	23,000	.26	1090	693.2
River Mile 11.8*	50%	25.075	24,500	1.0	1100	694.6
St. Mary, WV 03115000	90%	15,560	18,000	.8	1045	567.1
River Mile 155.0	50%	44,540	19,500	2.3	1060	568.2
Parkersburg, WV 03151000	90%	7,830	19,500	.4	1370	572.5
River Mile 184.4	50%	29,570	33,000	.9	1400	583.5
Point Pleasant, WV 03201500	90%	32,500				
River Mile 265.4	50%	84,800				
Huntington, WV 03206000	90%	11,560	32,200	.36	1360	515.0
River Mile 311.6	50%	46,060	34,000	1.35	1370	516.2
Ashland, KY 03216000	90%	12,795	42,500	.30	1150	514.5
River Mile 322.5	50%	53.560	43,500	1.22	1170	515.5
Maysville, KY 03238000	90%	14,215	19,000	.70	1500	485.0
River Mile 408.3	50%	52,380	32,000	1.65	1600	486.0
Cincinatti, OH 03255000	90%	14,080	33,500	.42	1075	455.6
River Mile 470.5	50%	59,000	34,000	1.75	1100	456.6

Ohio River , continued

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Louisville, KY 03294500	90%	13,850	15,000	.9	1400	383.4
	50%	62,600	23,000	2.8	1700	387.5
River Mile 607.3						
Evansville, ID 03322000	90%	19,360	34,000	.57	1650	338.0
	50%	73,475	38,000	1.95	1730	340.6
River Mile 792.3						
Colconda, IL 03384500	90%	26,430	(38,000)	(.7)	(1500)	(303.1)
	50%	107,380	46,500	2.3	1750	308.6
River Mile 903.1						
Metropolis, IL 03611500	90%	11,180	(68,000)	(.15)	(2700)	(290.8)
	50%	102,300	78,000	1.3	2800	291.8
River Mile 944.0						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						
	90%					
	50%					
River Mile						

* Miles from Pittsburg, PA

Okeechobee Waterway

The Okeechobee Waterway connects the east and west sides of Florida by traversing the St. Lucie River Canal, Lake Okeechobee and the Caloosahatchee River. Distances along the Okeechobee Waterway are numbered from St. Lucie Inlet (mile 987, AIW).

Project depth for the Waterway is 8 feet from the St. Lucie to Fort Myers on the Gulf side. Flow out of Lake Okeechobee is controlled by dams and locks at mile 15 on the St. Lucie River and miles 77, 93, and 122 on the Calooshahatchee River. Total mileage for the waterway is 140 miles.

Stage I

The Okeechobee Waterway is unusual in that more than half of the time there is no flow out of the lake except for leakage through the locks. The median flow is then quite small and difficult to determine accurately. In the case of the gaging station at Stuart, 61% of the time the flow was greater than 10 and less than 15 cfs. A similar situation existed at Moore Haven.

A class 4 rating was assigned the St. Lucie to mile 39 and the Caloosahatchee River from mile 77 to 140. The section of the waterway through Lake Okeechobee from mile 39 to 77 was assigned a class 1.

Stage II

Due to the extremely low flow velocities, the spill volumes required to produce 1,000 ppm 25 miles downstream are somewhat misleading. The values are presented for uniformity. Taking D as given by 20 u*h, as reported by Thomas on a similar dredged canal, gives the following values:

Mile	D	W/h	Tons
15	.3	37.5	226
77	.1	37.5	225

Okeechobee Waterway

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
St. Lucie Canal near Stuart, FL	90%		1,700			
02277000	50%	14	1,700	.008	300	
River Mile 15.0						
Caloosahatchee River at Moore Haven, FL	90%					
02292000	50%	42	1,700	.025	300	
River Mile 77.0						
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					

Penobscot River to Bangor, Maine

The Penobscot River to Bangor is open to moderate draft vessels. The tides are quite strong, with a mean tidal range of 11.0 feet at Bucksport and 13.1 feet at Bangor.

Stage I

No tidal current information was available for the river above Bucksport. Using tidal cubature, the tidal discharges in the accompanying table were computed. These values indicate a class 1 rating.

Stage II

Tidal excursion is 3.6 miles at Bucksport where 32,200 tons are required for a 1000 ppm concentration in the tidal excursion. At mile 14, the tidal excursion is 2.1 miles where 6,300 tons are required.

Penobscot River to Bangor

LOCATION	AREA ft^2	TIDE VEL fld	TIDE VEL ebb	Q cfs
Mile 0	99,000			*
Bucksport (Mile 3)	61,200			52,200
Mile 9	30,000			26,800
Mile 14	20,100			9,900
Mile 19, Bangor, Maine	4,500			
West Enfield (50% Q) (40 miles above Bangor)	3,600			7,600

*by tidal cubature

Sacramento and San Joaquin Rivers

The Sacramento River is navigable from Colusa, CA, to Sacramento and on to Suisun Bay. From Sacramento on to the bay, however, the Sacramento Ship Canal has been dredged providing a 30 foot deep channel. Flow from the river into the channel is regulated by a lock at Sacramento.

Navigable length of the Sacramento River to Suisun Bay is 145 miles. Project depth is 6 to 10 feet and width is up to 300 feet.

The San Joaquin River is navigable from Hills Ferry, CA, to Suisun Bay, a distance of 127 miles. From Stockton on to Suisun Bay, the channel has been dredged for ocean-going vessels (30 feet). Above Stockton, depth is less than 6 feet while width is 225 to 400 feet.

Stage I

The Sacramento River is partly in classes 1 and 2. Because the majority of the length of the stream has a median flow less than 10,000 cfs, a class 2 rating was assigned.

The San Joaquin River is partly in classes 2 and 3. A class 3 rating was assigned.

Stage II

Dispersion measurements on the Sacramento River are reported in Fischer (1973) although there is insufficient information presented to determine the part of the river in which measurements were made or what the flow conditions were. The rather low value for D (150 ft^2/sec) is indicative of a well maintained channel with few dead zones or flow irregularities. This description fits the navigable portion of the Sacramento very well. The prediction relation of Fukuoka and Sayre yields somewhat higher values, although they agree quite well at some gaging stations. The McQuivey and Keefer values are also presented, but are in general two orders of magnitude too large.

Since the river characteristics change considerably during its length, the one measurement of 150 ft^2/sec was not applied over the whole length. Instead, the F and S values were employed and the reported measurement used as confirmation. Values for r_c and L were 2,592 and 7,776 feet, respectively.

No dispersion measurements are reported for the San Joaquin River. Using the method of Fukuoka and Sayre with r_c and L taken from charts as 2,215 and 6,070 feet, respectively, the following spill amounts required to produce 1,000 ppm 25 miles downstream were calculated for the Sacramento River:

Mile	D_F and S (ft^2/sec)	W/h	Tons	D_M and Q (ft^2/sec)
89.4	875	35.6	1,730	17,360
62.9	515	20.4	1,360	28,990
34	285	12.4	1,410	30,600
19.6	908	48.1	3,730	20,100
0	856	50.0	5,680	

D reported in Fischer (1973), 150 ft^2/sec

Calculations for the San Joaquin River were:

Mile	D	W/h	Tons
(120)	918	88.9	441
(40)	794	56.5	957

Sacramento and San Joaquin Rivers

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Sacramento River at Colusa, CA	90%	5,130	2,000	(2.3)	300	38.3
11389500 River Mile 89.4	50%	7,970	2,700	2.9	310	40.3
Sacramento River at Grimes, CA	90%	4,600	(1,900)	(2.4)	(210)	(23.5)
11390500 River Mile 62.9	50%	7,630	2,700	2.7	235	27.0
Sacramento River at Knights Landing, CA	90%	5,210	(2,900)	(1.7)	208	(13.3)
11391000 River Mile 34.0	50%	8,400	3,500	(2.4)	208	15.9
Sacramento River at Verona, CA	90%	4,850	(2,900)	(1.7)	450	(8.3)
11425500 River Mile 19.6	50%	12,100	5,200	2.4	500	11.7
Sacramento River at Sacramento, CA	90%	9,020	(5,600)	(2.0)	580	(2.9)
11447500 River Mile 0.0	50%	15,700	(7,800)	(2.2)	625	(4.5)
San Joaquin River at Newman, CA	90%	164				
11274000 River Mile (120)	50%	540	450	1.3	200	8.5
San Joaquin River at Vernalis, CA	90%	605	440	1.3	210	8.5
11303500 River Mile (40)	50%	1,986	1,200	1.7	260	11.4
	90%					
River Mile	50%					

St. Johns River

The St. Johns River is navigable from Lake Monroe at Sanford, FL, to its mouth at Jacksonville, a distance of 152 miles. The navigable channel crosses several large lakes connected by relatively narrow stretches of river.

Tidal influence is strong in the lower reaches from Palatka (mile 57) to its mouth. Gaging stations are located at Christmas, FL, above the head of navigation, DeLand, FL (mile 123), Palatka and Jacksonville.

<u>Stage I</u>

For the first 100 miles above Jacksonville, the St. Johns is primarily tidally influenced. The average discharges are indicated by the following tidal current and USGS data:

Mile	Area	Vel*	$\overline{Q}$
30 (above Jacksonville)	116,100	.6 .9	60,400
57 (Palatka)	25,000	.4	10,000

*Average of USGS data. Velocity varied from +.8 to -.64 fps

At mile 88, the St. Johns enters Lake George. Tidal range in the lake is less than 0.5 feet and is primarily controlled by wind and flood conditions.

Above mile 100, the flow is entirely freshwater and is much smaller as indicated by the stream gaging data at DeLand, FL.

	Q	Area	Vel	Width
90%	950	6,200	.15	330
50%	2,550	6,300	.4	330

From mile 100 to 152, a class 2 rating is indicated, while for 0-100 miles, a class 1 rating is assigned.

Stage II

On the lower section, the amount of spill required to produce 1,000 ppm in the tidal excursion was estimated as follows:

Mile	Tidal Excursion	Tons
30	2.2	38,500
57	2.5	9,500

Using the dispersion coefficient prediction method of Fukuoka and Sayre, with r_c and L 1,436 and 3,890 respectively, the spill size required to produce 1,000 ppm 25 miles downstream was determined.

Mile	D	W/h	Tons
123	44.0	17.4	2,440

Savannah River

Navigation extends from its mouth to Augusta, Georgia, a distance of 215 miles. The river is regulated by New Savannah Bluff Dam (mile 203), which forms a navigation pool to Augusta and regulates flow for natural channel. Project depth is 9 feet and width is 90 feet. Navigation to Savannah is open to deep draft (38-foot) vessels.

The Savannah River is gaged by the USGS at three locations. In addition, a time of travel study was conducted by the South Carolina USGS District Office at mile 155.

Stage I

From Savannah to Augusta, 50% discharge indicates a class 2 rating (10,000 - 1,000 cfs). Below Savannah to the ocean, tidal motion rapidly increases available diultion capacity.

Stage II

With a time of travel study only slightly greater than 50% flow, a dispersion coefficient of 988 ft^2/sec was calculated. Using $n = .035$, and $u^* = .21$, this gives $D/hu^* = 497$ which for $w/h = 39.4$ is a reasonable value.

Amounts of spill required to produce 1,000 ppm concentrations 25 miles downstream below New Savannah Bluff Dam are, at the two gaging stations:

Mile	Ton
129.2	2,736
65	3,786

In the tidal section of the river from Savannah to the mouth, the following tidal current discharge values were computed.

Location	Cross-Sectional Area	Tidal Currents Knots		Q
		fld	ebb	cfs
Lower Savannah Harbor	18,200	2.4	3.5	43,270
Barnwell Island	31,000	2.0	2.8	60,000
Mouth of Breakwater	60,000	1.6	2.6	101,000

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		fld	ebb	cfs
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Barnwell Island	31,000	2.0	2.8	60,000
Mouth of Breakwater	60,000	1.6	2.6	101,000

Savannah River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Augusta, GA 02197000	90%	5,200	(9,200)	(.56)	660	101.0
River Mile 203.0	50%	6,500	(10,000)	(.65)	680	102.2
Burtons Ferry, GA 02197500	90%	5,900	(2,900)	(2.0)	(360)	57.8
River Mile 129.2	50%	7,700	(3,500)	(2.2)	(370)	59.5
Clyo, GA 02198500	90%	6,300	(3,800)	(1.6)	(330)	18.3
River Mile 65.0	50%	8,600	(4,500)	(1.9)	(340)	19.1
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					
	90%					
River Mile	50%					

Tennessee River

The Tennessee River is formed at Knoxville, Tennessee, and flows 652 miles to its confluence with the Ohio at Paducah, Kentucky. Project depth is 9 feet and width is 300 to 500 feet.

A series of 9 dams regulate the channel and provide hydroelectric power. Large lakes are formed behind these dams, notably Watts, Nickajack, Guntersville and General Joe Wheeler, but the gaging stations are located in reasonably representative areas of the streams.

Taking Manning's n as 0.03, and using the dispersion coefficient method of Fukuoka and Sayre (1973), the following values for D, and tonnages required were computed.

Average bend radius and length taken from navigation charts were 3,430 and 9,770 feet respectively.

Mile	D	W/h	Tons
651.4	486	51.5	12,090
429.7	404	26.9	14,600
333.6	1,390	97.6	28,000
256.7	1,360	81.6	25,800
189.9	594	24.5	12,700
	880	48.0	24,700

Tennessee River

GAGING STATION		Q cfs	AREA ft^2	VEL fps	WIDTH ft	ALT ft
Knoxville, Tenn. 03497000	90%	4,590	(11,500)	(.4)		(807.4)
River Mile 651.4	50%	11,650	13,700	.85	840	813.0
Hales Bar near Chattanooga, Tenn.	90%	15,030	20,300	.75	740	594.7
03570000 River Mile 429.7	50%	28,250	22,000	1.25	770	596.5
Whitesburg (Decatur), Ala.	90%	16,600	25,500	.65	1,500	(551.5)
03575500 River Mile 333.6	50%	34,460	24,600	1.4	1,500	553.2
Florence, Ala. 03589500	90%	19,320	(22,000)	(.85)	(1,370)	410.6
River Mile 256.7	50%	37,200	24,000	1.60	1,400	411.7
Savannah, Tenn. 03593500	90%	24,690	19,000	1.3	700	306.5
River Mile 189.9	50%	40,000	20,000	2.0	700	308.0
Paducah, Ky. 03609500	90%	29,000	27,000	1.05	1,150	
	50%	47,830	29,000	1.65	1,180	

APPENDIX B

The following is a practical example of how dilution capacity ratings might be used to manage water pollution risk from the bulk transport of oil and hazardous materials.

The Coast Guard may determine that, where the possibility exists of a spill of one-tenth of a barge's cargo that would result in more than twenty five miles of a river being essentially killed, additional safety precautions would be required.

Stage I or II dilution capacity ratings could be used in conjunction with the NAS acute toxicity ratings to provide a measure of when this condition exists. For example, a substance with a NAS aquatic toxicity rating of 2 (LC_{50} between 10 and 100 ppm) would theoretically produce the critical impact area if between 10 and 100 tons of material were spilled into a stream whose Stage II rating was 2.0. Since the typical tank barge has a capacity of between 1,000 and 3,000 tons, this is well within the one-tenth capacity figure.

Given the criteria of a 25-mile impact zone resulting from a one-tenth capacity spill, a reasonable estimate of the existence of the situation would be obtained if the sum of the two ratings were greater than four. Where this sum was greater than 4, additional spill reduction measures would be required.

The additional spill reduction precautions could either reduce the probability of an accident or reduce the likelihood of release should the accident occur. These precautions could be any of a number of options, including: use of double-hulled or specially compartmented barges, operation in daylight only, operation only under special escort, or rafting of the OHM barge in the middle of a tow.

Example Situation

A shipper is transporting acrylonitrile from New Orleans, LA, to Belle, WV. The NAS aquatic toxicity hazard rating for the substance is 2. For the Mississippi and Ohio Rivers portion of the trip, the combined hazard ratings are lower than four, so that only normal shipping measures are required. The sum of the ratings on the last portion of the trip up the Kanawha River to Belle is greater than four, however, requiring one of the additional safety precautions.

The shipper could elect to use a double-hulled barge for the entire trip. This, however, would entail considerably greater expense because of the initial cost and the reduced useful payload of the double-hulled barge. A more likely option would be to reduce the probability of accident through navigating the last portion of the Kanawha River during daylight hours or with a special escort, if time is important.

APPENDIX C

Another application of the results of this study is use by transportation planners to estimate appropriate degrees of safety for a particular aquatic system.

For example, what degree of safety would be required for the carriage of phenol, with an assumed toxic threshold of 10 ppm (NAS aquatic toxicity rating of 2)? The route is from Savannah to Augusta, GA, via the Savannah River.

By consulting Table 6.2, it can be seen that 2,700 tons of material spilled would produce a 1,000 ppm concentration twenty-five miles from the spill site. In the case of phenol, 27 tons would produce approximately the twenty-five mile area of damage.

Furthermore, area transportation planners have decided that a spill which severely damages aquatic life for twenty-five miles is too serious to accept. A consensus is reached that only one-tenth that distance could be tolerated without major modification of the area ecosystem. From Figure 5.2, it can be determined that the allowable spill size must be reduced to 31.6% of the 27 tons, or 8.5 tons in order for the impact area to be limited to 2.5 miles.

The tonnage arrived at in this manner is an indication of the spill size that can be tolerated for a system with the stated assumptions. A reasonable approach from this point would be to limit probable spill size to below the determined amount. This could be done either by limiting container size, requiring interior subdivision of the container, or self-sealing devices in the tanks. Another alternative would be to not limit spill size but rather reduce the probability of an accident by such techniques as special escort and advance notification of transit.